CIVIL ENGINEERING

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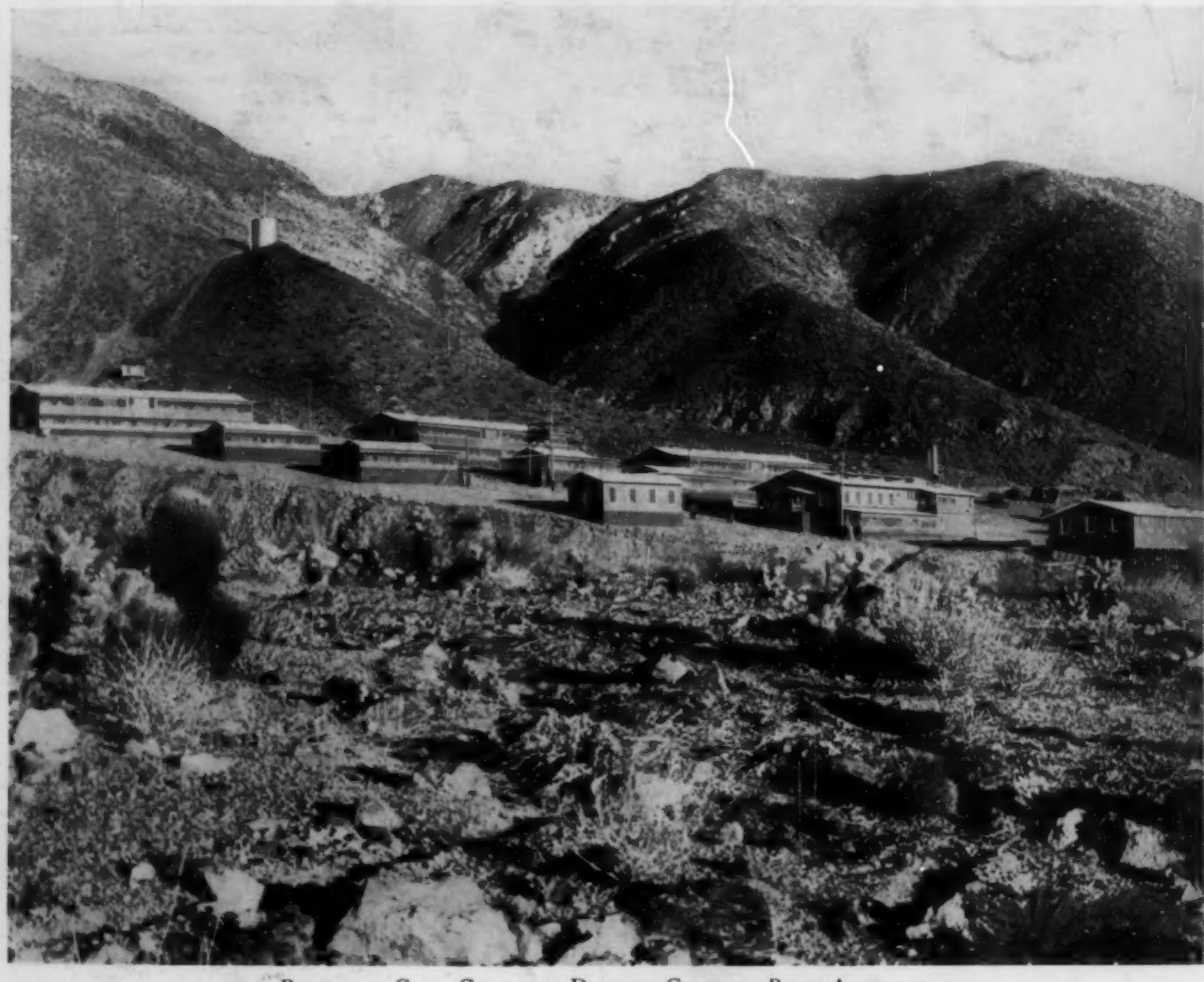

Pushwalla Camp, Coachella Division, Colorado River Aqueduct

Volume 4 — AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 — *Number 5*

MAY 1934

Among Our Writers

C. W. Gennet, Jr., has made a specialty of inspecting and testing railroad materials. For 20 years he was with the Robert W. Hunt Company, inspecting and testing steel rails. Since 1928 he has been in charge of testing rails in track for the Sperry Rail Service Corporation.

Robert S. Stockton, a graduate of the Colorado School of Mines and one-time Professor of Mathematics and Surveying at that institution, was an engineer in the U. S. Reclamation Service from 1903 to 1911. Since then he has been in charge of the Western Section Irrigation System for the Canadian Pacific Railway Company in Alberta.

F. E. Weymouth from 1902 to 1924 served the U. S. Bureau of Reclamation—from 1916 to 1920 as chief of construction, and from 1920 to 1924 as chief engineer. In 1924 he became chief engineer of the J. G. White Engineering Corporation in Mexico. In 1929 he became connected with the Colorado River Aqueduct project.

Wallace M. Lansford received the degree of B.S. in civil engineering from the University of Illinois in 1924. For the next 3½ years he was engaged in municipal engineering work. In 1927 he returned to the university for graduate study and in 1929 received the degree of M.S. and in 1931, that of C.E. Since 1928 he has been employed by the university, first as Special Research Assistant, and for the past 5 years as teacher of mechanics and hydraulics.

M. M. O'Shaughnessy has been engaged in civil engineering practice for 49 years. He has spent 5 years in the Hawaiian Islands building aqueducts; 5 years in San Diego, completing, among other projects, the Morena rock-fill dam; and 20 years in San Francisco as city engineer, completing the Municipal Railways, and the Stockton, Twin Peaks, and Sunset tunnels. At present he is consulting engineer to the Public Utilities Commission of that city.

Blair A. Ross, since graduation from the University of Missouri in 1912, has spent over 16 years on the Mississippi River and its tributaries between Cairo and the Gulf, engaged in levee and bank protection work and flood control studies.

Pao-Tse Sun, after graduation from Chiao Tung University, Shanghai, in 1918, came to the United States and worked for the American Bridge Company for over two years. From Harvard he received his M.S. in C.E. in 1921. After returning to China in 1922 he taught successively at Tangshan Engineering College and Central University, Nanking. He was assistant bridge engineer of the Kiao Chow Tsinan Railway for over three years and became bridge engineer in 1930.

F. L. Blue, Jr., a 1929 graduate of West Point, in that year entered the U. S. Engineer Office in Wilmington, N.C. In 1930–1932 he was with the 13th Engineers in Virginia and Kentucky. In 1932–1933 he was detailed to the University of Iowa for postgraduate study in hydraulics. He is now at the Engineer School, Fort Humphreys, Va.

J. K. Herbert graduated from the U. S. Military Academy in 1930 into the Corp of Engineers, in which he served two years as assistant to the District Engineer in the Boston District. In 1932–1933 he was detailed to study at the University of Iowa, whence he received the degree of M.S. At present he is with the 13th Engineers, at Fort Humphreys, Va.

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Defects in Railroad Rails

Probable Causes, Means of Detection and Elimination of Fissures Discussed

By C. W. Gennet, Jr.
Member American Society of Civil Engineers
Vice-President, Sperry Rail Service Corporation, Chicago, Ill.

Under existing methods of manufacture, a small percentage of railway rails contain invisible defects which under certain conditions of present-day traffic develop into failures. Although the problem is being intensively and continuously studied by engineers and experts of both the railroads and the rolling mills, much of the evidence uncovered is contradictory. In this article Mr. Gennet discusses probable causes for the failure of rails connected with their manufacture and use. While one group blames the impact of wheels and the intensity and frequency of the reversal of stresses caused by traffic, another finds sufficient cause for failures in the internal stresses set up in the steel itself by the rapid reduction and elongation of the high-carbon steel billet to rail size. Although rail defects invisible to the sharpest eye can now be located electrically by an ingenious detector car which is operated over the track, the reasons for their occurrence are still little understood.

A little over forty years ago, in 1893, the Society, after a careful study by a committee of prominent engineers, recommended the use of a series of rail sections. For twenty years these A.S.C.E. sections, as they are commonly called, constituted the principal American standards. Since that time other associations and groups of railroad engineers and metallurgists have given intensive study to rail problems. Changing conditions of rail manufacture and the new and greater burdens imposed on rails due to the tremendous growth of railroad business have demanded new and heavier sections, and changes in specifications for their manufacture. One of the most important problems at the present time concerns the frequent occurrence of that peculiar type of rail defect called "transverse fissure"—a complex problem to which experts representing both manufacturers and users of rails are applying themselves.

Indicative of the growth in railroad traffic is the fact that in 1902 there were in the United States 274,000 miles of track, including second, third, and fourth tracks on multi-track lines and all tracks in yards and sidings. By 1930 the total in these same categories had increased to 424,000 miles. Tracks had been extended into all sorts and types of country and had been laid on heavy grades and sharp curves in the Rockies and Alleghenies and on long, high-speed tangents on the plains. In 1902 there were 1.2 billion tons of freight carried on the railroads of the United States, and by 1926 this total had reached a maximum of 2.5 billions of tons.

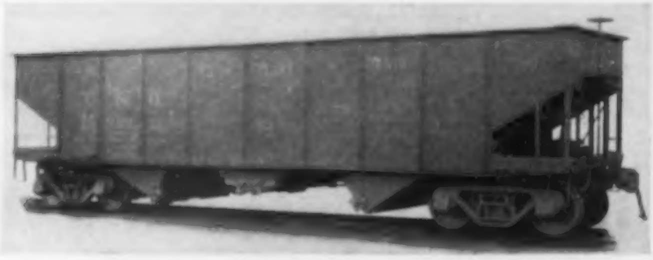
Density of traffic over the lines of different railroad companies varies widely. In 1930 it ranged from a million ton-miles of traffic per mile of track on one line to over six million ton-miles per mile of track on another line having but 2,200 miles of track.

The average weight of locomotives in 1902 was about 56 tons; in 1930 it had increased to 111 tons. Today wheel loads of 35,000 lb are common; a single driving wheel frequently exerts 40,000 lb of static pressure. The size and weight of freight cars have increased 70 per cent in the past 30 years. There are coal cars of the hopper-bottom type which have a

American Locomotive of 4-8-4 Type with 12-Wheeled Tender

rated capacity of 180,000 lb and themselves weigh 60,000 lb. With an allowable increase of 10 per cent over the rated capacity of the car, the load carried on one of the eight wheels may be 32,000 lb.

This increase in weight of both locomotives and cars has been coincident with increased speed in practically all classes of trains. The average speed of freight trains has increased 30 per cent in the past ten years. Today speeds of 80 and 90 mph are attained by passenger trains on short stretches of track. Of necessity rails have constantly increased in size and weight. Thirty years ago those weighing 80 lb per yd were in common use. Today on over 75,000 miles of track the rails weigh more than 100 lb, and on 20,000 miles they weigh over 125 lb, whereas some weigh as much as 152 lb.



HOPPER-BOTTOM COAL CAR WEIGHING 210,000 LB WHEN LOADED

STEEL RAIL MANUFACTURE

A brief picture of the manufacturing process will aid in an understanding of rail failures. For the past several years there have been eight steel plants in the United States with the necessary equipment for rolling the rails in general use. In some of these plants rails are the principal product but in others they are but one of many products. In each plant there is generally a special mill in which the rails are rolled. Such a mill, with its hot (or cooling) beds and its cold finishing departments, must lie idle when no rails are being produced.

Probably the outstanding feature of American blast furnaces is their huge size and capacity, which sometimes are sufficient to produce as much as a thousand tons of iron per day, though of course there are a number of smaller furnaces in operation as well. For nearly twenty years practically all steel for rails has been made by the basic open-hearth process. An open-hearth furnace generally produces heats, or melts, of from 90 to 125 tons of steel, and consequently large units such as ladles and ingot moulds are necessary. Ingots are always top cast in moulds, setting on cast-iron stools, with the large end down. As a rule no effort is made to keep the small upper end hot in order to minimize piping and segregation. Ingots weighing from 10,000 to 15,000 lb are common, the dimensions of the large end being about 2 ft square, and the height of the ingot from 6 to 7 ft. A single ingot may make from 5 to 10 rails, each 39 ft long, depending on the weight of the rails being rolled, and a single heat of steel may consist of from 18 to 25 ingots. It is not uncommon for 100 or even 150 rails to be rolled from one heat of metal.

Ingots are charged into gas-fired soaking pits of the regenerative type, utilizing either coke or producer gas. As a general practice, the time between casting a heat and charging its ingots into the soaking pits averages from one to two hours, and the time that the ingots are in the soaking pits averages slightly over two hours.

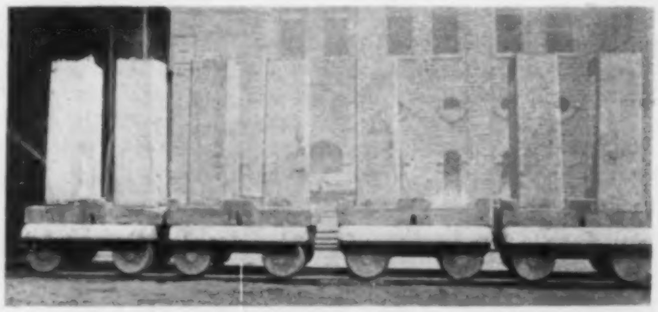
From the soaking pits the ingots are fed directly into either two or three roll-high blooming mills, where they are given from a minimum of 9 to as many as 25 passes, depending on the particular mill, to form blooms 8 to 10 in. square. These blooms are frequently reheated in gas fired furnaces and then passed to the rail mill proper, where in from 9 to 13 passes they are reduced to the section and weight of rail desired.

After the rails are hot sawed to standard lengths, generally of 39 ft, they pass through hot cambering rolls on to hot beds for cooling. When sufficiently cool to be easily handled they are cold straightened in a gag press, the fixed supports of which are at least 42 in. apart. The necessary bolt holes are then drilled in the ends, and the rails are inspected and loaded for shipment.

RAIL FAILURES AND FISSURES

In spite of the combination of large volume of traffic, heavy wheel loads, high speed of trains, and severe climatic conditions, rail failures have steadily decreased over a period of years, according to statistics of the American Railway Engineering Association. Based on records of five years of service, the rate of failures has decreased in the past 16 years from more than 200 per 100 track miles to approximately 110.

There can be little doubt but that the rate of failures frequently reflects the practice of the individual mill at which the rails are rolled. Statistics of the American Railway Engineering Association show that rails of some mills are more susceptible to failures than are those from others, and such differences are not easily explained, if at all, by usual track and traffic conditions. Naturally therefore, the actual practice that a mill employs in producing rails must be in some way responsible. Unfortunately it appears impossible to determine the particular phase of manufacture that, over a period of time, may have been the cause, since so many variables enter into the manufacture of pig iron and steel and its ultimate rolling into rails.



OPEN-HEARTH STEEL INGOTS EN ROUTE TO SOAKING PITS

In general the principal feature of present-day rails is the chemical composition of the steel; that commonly used is called "high carbon." Carbon content, while as low as 0.60 to 0.75 per cent for rails up to 90 lb per yd, may be from 0.75 to 0.89 per cent for 130-lb rails. These ranges for carbon have been dictated entirely by the necessity of prolonging the life, or wearing qualities, of the rails, especially in curved track.

Although there are a number of types of rail failures, those known as "head failures" invariably predominate and comprise close to 50 per cent of the total. The common type of head failure is a vertical or horizontal split which extends rapidly in the general plane of the direction mentioned. Transverse fissures, which extend in a vertical plane, crosswise of the rail, generally are classified separately.

In the case of vertically split heads, an interior cavity of considerable size develops lengthwise of the rail,

with a corresponding widening of the rail head and frequently some breaking down in the immediate locality of the fissure. Heads split horizontally appear to have much the same character as those split vertically, but very often in some details they resemble typical cases of transverse fissures. These horizontal splits, usually starting in the metal about $^1/_2$ in. beneath the running surface of the rail, develop lengthwise and sidewise, often exhibiting rings indicative of a progressive growth somewhat similar to that of transverse fissures. This lateral growth causes them eventually to reach the surface, where they manifest themselves as hair cracks. Later there may be some slight breaking down of the metal, and a small lip may form. Splits of this kind have been found from very small size up to a foot or more in length. In their infancy they have a bright silvery luster but later, when they extend through to the outside of the rail, air is admitted and their surfaces become dark.

RAILS MARKED WHEN ROLLED

Common practice requires that each rail have stamped on it not only its heat number but also a letter indicating its position in the ingot, "A" being the top or first rail made from the ingot, "B" the second, and so on. In addition to the heat number and rail letter, each rail bears a number indicating the ingot from which it was poured. While this last number may not show the order in which the ingots were cast, it nevertheless provides a means for identifying the rails from any particular ingot.

Heads split both vertically and horizontally occur to a remarkable extent in the rails rolled from the upper parts of the ingots. Although transverse fissured rails are quite evenly distributed, almost always more split heads are found in "A" and "B" rails than in rails from lower down in the ingots. It has also been noted that the amount of the "top" discarded from ingots may bear a distinct relation to split heads. Chemical and etching tests have further indicated rather clearly that chemical and physical unsoundness is a cause of these splits, and in general the evidence plainly points to the conclusion that segregation and physical unsoundness of the metal in the top of the ingot are associated with split heads of both types. Further it appears that "hard" heats, or those of higher carbon content, are more likely to produce horizontally split heads, whereas vertical splits may predominate in steel of lesser hardness and consequent greater ductility.

TRANSVERSE FISSURES HAVE DISTINCTIVE CHARACTER

Except for the small nucleus of rather rough granular metal, the entire surface of a transverse fissure is smooth, and not crystalline as might be expected of any ordinary fracture in steel. Frequently there is a pronounced series of more or less concentric rings about the nucleus indicative of the progressive growth of the fissure.

All are agreed that transverse fissures originate in a minute nucleus, from which, under the strains of traffic, the resulting interior crack develops. Therefore the establishment of a definite cause for the existence of that nucleus is of primary importance in order that means may be taken for its prevention and elimination. Various chemical, physical, and microscopic tests have consistently failed to establish the presence of any abnormal chemical condition or of any impurity that might account for the nucleus. But certain etching tests have revealed a somewhat "shattered" condition of the steel in the head of fissured rails. This condition, apparently free from the complications to which foreign impurities might contribute, has been rather definitely ascribed to abnormal stresses set up in the metal, which for some reason it is unable at times to sustain. The result is a minute interior rupture at some point of maximum weakness. Such a small rupture may be the nucleus from which traffic later develops a real transverse fissure.

One school of thought definitely fixes the reason for this shattering as the stresses resulting from the wheels of traffic. The adherents of this point of view assert with considerable force that very heavy wheel loads, together with high frequency and concentration, cause such tremendous compressive stresses at the surface of the rail head that the steel in a zone beneath is unable to resist them and the counterbalancing effect of tension,

SELF-PROPELLED DETECTOR CAR FOR LOCATING AND RECORDING DEFECTS IN RAILS

with the result that minute rupture of the metal occurs. The other school of thought regards the "shattering" as more probably due to interior stresses caused by the thermal changes to which the metal is subjected in its progress through the mill from ingot to finished rail. There is an abundance of evidence in support of this reasoning also, for it can scarcely be denied that high carbon steel undergoes severe treatment in being rapidly shaped to a rail section and then permitted to shrink 8 in. in length to make a 39-ft rail at atmospheric temperatures.

Transverse fissures have occurred at an alarming rate on American railroads in the years since attention has been so persistently drawn to the subject. Published statistics show that approximately 7,500 fissured rails were found on main-line tracks in the United States and Canada in the year 1930. Necessarily this high total, coupled with the fact that in many cases the defect is so completely concealed in the rail as easily to pass the most careful inspection by track walkers, has caused fissures to be considered the most serious menace to the safety of tracks. With no visible indication except a hair-line crack on the surface—generally on the side of the rail head—and perhaps a rust mark on the web, before 1928 the chances were extremely remote that fissured rails would be located prior to their actual rupture, in which event the automatic signal system might become operative and prevent derailment of trains.

DETECTING FISSURES ELECTRICALLY

Realization of the gravity of this situation led Elmer A. Sperry to invent the detector car, which since late in 1928 has been in common use on American roads as the principal means of locating transverse fissured rails in track. Sperry detector cars are equipped with a direct-current generator supplying a high-amperage, low-voltage current. By means of two brushes about 4 ft apart, bearing on the running surface of the rail on both sides of

the track, it is possible to energize the rails as the car moves along them. Thus a rather extensive magnetic field is set up, which under normal conditions is practically constant. Midway between the two brushes is a detector connected to amplifiers, relays, and a recording unit. Thus when a defective rail is encountered the field is distorted, and the impulse created by the defect, after being multiplied many times, is automatically recorded

TRANSVERSE FISSURES IN A RAIL
Left, Well-Defined Nucleus and Annular Rings Indicating Progressive Development
Right, Fissure Spread to Side of Rail Head, Showing Changed Color Due to Admission of Air

on a very definite and accurate pen graph, which serves as a lasting log of the test.

Paint guns in the circuits are also actuated by the defect so that a daub of white paint is put on the rail at the exact location of the defect. The operator at the recording table watches the record and the track carefully and is able by experience and practice to interpret various indications made by the pens on the record. Occasionally he stops the car, which is traveling at a speed of about six miles an hour, and backs it up to make a confirmatory test. At times he resorts to a "hand test," made while the car is stationary, by which he can very accurately determine the size and location of a transverse fissure.

Various surface defects in rails, such as a burned spot made by slipping wheels, flat spots, and flaws, give indications on the record much the same as do interior defects of the fissured and split-head types. But the operator's ability to differentiate between the causes for the indications makes it easy to separate the defects found into the various classes desired. Thus it is possible with detector cars definitely to locate interior defects long before any manifestation of them has appeared on the surface of the rail. No one can overestimate the great importance of this method of locating defects, for the protection against serious derailment of trains is greatly increased by the ability to remove defective rails from the track as fast as the detector car points them out.

TABLE I. SUMMARY OF RESULTS OF DETECTOR-CAR TESTING IN THE UNITED STATES AND FRANCE

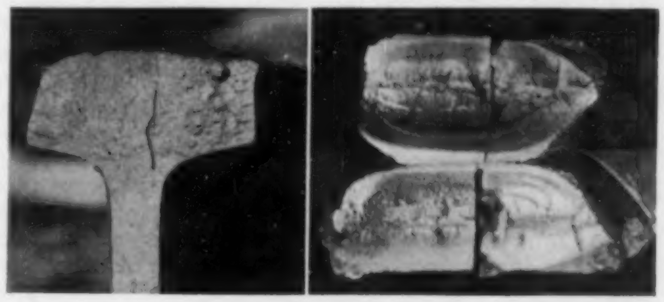
ITEM	UNITED STATES	FRANCE
Miles of track per transverse fissured rail found	5.8	7.7
Miles of track per horizontally split head found	9.7	10.1
Miles of track per vertically split head found	4.5	6.2
Miles of track per defective rail of all other kinds found	77.1	532.1
Miles of track per defective rail of any kind found	1.9	2.5
Percentage of transverse fissured rails to all defective rails found	33.7	33.0
Percentage of horizontally split heads to all defective rails found	20.2	25.3
Percentage of vertically split heads to all defective rails found	43.5	41.2
Percentage of other defective rails to all defective rails found	2.6	0.5
Percentage of fissured rails found known to contain more than one fissure	12.1	33.6

A detector car will test from 15 to 30 miles of track in an 8-hr day; the average is a little over 20 miles a day. Variations of course are caused by delays due to traffic conditions and frequently by the character of the rails being tested. All the cars are self-propelled and are serviced and maintained by their crews, consisting of three men.

One of these cars has been operating in France for several months. The results of such tests on American roads are made still more interesting by comparison with statistics obtained in France. The data in Table I are based on the testing of about 30,000 miles of track recently covered in America and about 1,000 miles in France. Most of the American rails were of open hearth steel, but in France basic Bessemer steel predominated.

HOW EXPLAIN FISSURES?

Apparently no mill turns out rails that are immune from the development of transverse fissures. Although it is perhaps true that the product of some mills shows a lower percentage of fissures than that of others, differing traffic conditions are apt so to cloud the records as to make it impossible accurately to determine the relative standing of the various mills. Neither the actual manufacturing process, the ore, the iron, nor the method of steel-making appears to be of more than ordinary importance in connection with the occurrence of this defect. Further, it appears plain that fissures develop in rails laid under all sorts and conditions of track and traffic. They are found over and between ties and in track laid on all kinds of ballast, on bridges and in tunnels, so that



TYPICAL SPLIT HEADS
Left, a Head Split Vertically; Right, a Horizontally Split Head After Being Uncapped, Showing a Transverse Component of the Crack Developing at the Left End

it appears that elasticity of track is not necessarily an important factor, although it may increase the rate at which fissures grow. Neither tangents nor curves seem to make any difference, and the importance of grades is probably negligible except as they may influence the speed of traffic and consequently, again, the rapidity with which the defect grows.

If the steel is susceptible to fissures, they will develop under all kinds of conditions; but even so, the character of traffic naturally plays a part. Lines on which there is a dense traffic certainly suffer from fissures more than those on which the traffic is lighter. Probably the frequency of wheel loads, and certainly speed, are factors. Fissures are known to have occurred in rails of subway tracks where wheel loads are comparatively light, but where speed and frequency are correspondingly high. Tracks devoted almost exclusively to slow freight may not appear to show as high a rate of fissure occurrence as tracks where high-speed passenger trains are numerous, although in cases offering this kind of comparison other factors may enter that cast doubt on the results.

In one known case, a detector car tested nearly 200 miles of track and found a fissured rail every 4.6 miles.

Exactly 10 months later, when the same track was tested again by the same detector car, a fissured rail was found every 5.3 miles. Thus a new crop of fissures had developed in the interval although traffic on the track in that time had been nearly 30 per cent less than in the corresponding period before the first test. Plainly, not only was the steel susceptible to fissures but the new crop was more the direct result of speed, which had not been reduced, than of the amount of traffic, which had been much reduced.

MANUFACTURE AND COMPOSITION STUDIED

In considering the part played by the steel itself in the development of fissures, numerous enigmatical situations arise. First there is the outstanding fact, which apparently cannot be overemphasized, that some rails are much more susceptible than others to this defect. The product of a single mill for a certain period of time may amount to a hundred miles or so of rails, which may be shipped to one or more roads. Heat after heat of those rails over a period of years may be absolutely free from fissures. But rails from another heat, perhaps from the same furnace and rolled in the mill at the same time, may suddenly begin to develop fissures. Those fissured rails may be separated, through the vagaries of distribution and track-laying requirements, by wide distances and they may even be on different railroads. Several such instances have occurred. Naturally they lead to the assumption, if not the conclusion, that various heats of steel are susceptible to the occurrence of fissures while other heats are practically immune from them. It is absolutely impossible to predict the guilty heat when rolled, for time and traffic apparently offer the only method of testing. The caprices of certain heats are illustrated in Table II.

Table II. Data Showing Disposition of Some Heats to Develop Fissured Rails More Than Others

Heat No.*	No. Ingots Cast from Heat	No. Ingots from Which Fissured Rails Developed	No. Rails Shipped	No. Rails Showing Fissures	Composition of Steel % C	Composition of Steel % Mn
A	24	2	142	2	0.82	0.66
B	25	17	121	28	0.75	0.75
C	27	2	155	2	0.82	0.80
D	26	16	150	37	0.82	0.80
E	26	15	155	35	0.74	0.79
F	24	13	141	20	0.81	0.78
G	22	19	110	56	0.88	0.85
H	20	16	99	46	0.87	0.74
I	24	21	117	60	0.84	0.87
J	24	18	137	41	0.81	0.78

* In order of drawing from the same furnace.

One point fairly definitely indicated by various records and statistics is that the carbon content of the steel is an important factor. Heats in which the carbon content is high appear to be more prone to the development of fissures than others, but even this is by no means definitely proved. Possibly other elements than carbon, perhaps manganese and silicon, combine in some way to encourage fissures in certain heats, for some of the results seem to show that hard heats, rather than softer ones, may be commonly expected to produce the greater number of fissured rails under traffic.

Another strange and perhaps incidental phenomenon is that rails rolled in winter months may be more prolific of fissures than those rolled in summer. This is difficult of proof but at various mills the weather conditions are at times severe, and it does not seem unreasonable to suppose that chilling winds and low temperatures of winter months may have an effect on the cooling of the rails of high carbon steel.

Apparently rails can contain any number of transverse fissures, some remarkably close together and in transverse planes so minutely separated that two fissures on the fractured section appear as twins. There are of course a great many cases of a single fissure in a rail length. On the other hand, there is on record a case of a 39-ft rail which when carefully broken up was found to contain 135 distinct fissures. Rails having more than one known fissure are called "multi-fissured," and in all probability are much more common than the records would indicate. Necessarily a multi-fissured rail is considered to present a greater hazard as the number of known fissures in it increases.

SOLUTION OF PROBLEM BESET WITH DIFFICULTIES

From this review of some of the facts regarding the origin and development of transverse fissures, it must be plain why they constitute such a baffling problem. Indeed there are so many variables present that the moment one important clue appears to have been found, some other contradicting and offsetting condition is revealed which again clouds the issue. The menace of fissured rails to American transportation cannot be overestimated, but certainly there are sufficient reasons for the inability of metallurgists and others to clear up the perplexing question of their origin. Yet progress is constantly being made although it often seems slow and discouraging. In recent months it has gained momentum because of new cooperative studies that are being conducted by a joint committee of rail manufacturers and railroad engineers.

A Typical Single-Track Railroad

Maintenance of a Canadian Irrigation Flume

A 24-Year-Old Conduit Successfully Waterproofed with a Coating of Pitch and Tallow

By Robert S. Stockton

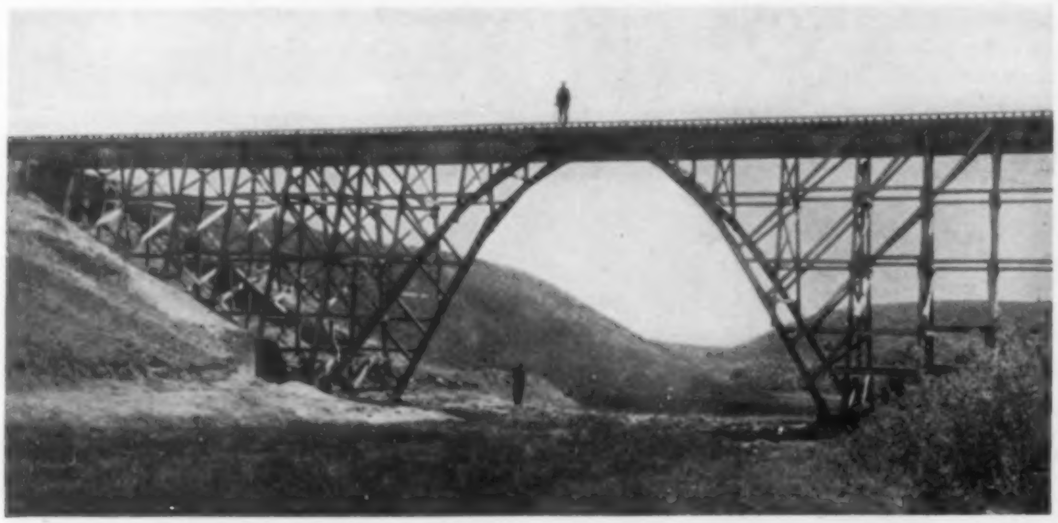
Member American Society of Civil Engineers

Superintendent of Operation and Maintenance, Western Section Irrigation System, Canadian Pacific Railway Company, Strathmore, Alberta, Canada

Built in 1910 without expansion joints, the 210-ft Winona Flume later developed leaks at the riveted joints of its galvanized plates as a result of movements due to the great temperature range. Softened ground, rotting timbers, frost heave, and undermined bents threatened the structure. In 1924 the upper section of supporting timbers was replaced by a wooden arch, and leakage was successfully stopped by an interior coating of pitch and tallow. This mixture, when applied hot, adhered to the metal and completely sealed all cracks. According to Mr. Stockton, this rubbery coating, when placed thick on the bottom of steep metal chutes, has proved economical and useful as a protection against abrasion and corrosion.

In 1910 the Canadian Pacific Railway Company built the Winona Flume to carry water across a coulee in Alberta. The story of this structure, which is part of the Western Section Irrigation System, covers the vicissitudes of 23 years of operation. The conduit was made of 16-gage galvanized sheets, 28 in. wide, riveted together with a lap of $1^1/_2$ in. to form a semicircular flume with a radius of 42 in. Pile bents supported the waterway, which was provided with a concrete inlet and outlet. The flume was 210 ft long and about 33 ft above the bottom of the coulee at its highest point. It was built with a fall of 2.33 ft and had a calculated capacity of 141 cu ft per sec. In 1911, water was first run through the flume, which has been in use ever since. However, many difficulties have developed with the years.

This was due to the fact that no provision was originally made for expansion and contraction. At the close of each winter the flume pulled loose at one end, and the riveted seams opened and leaked. The trouble continued until a bad break occurred. As a result of frost action, the pile bents were lifted and had to be cut off to grade several times. In 1924 the four central bents, which were in wet ground, were replaced by a timber arch of 64-ft span, and the remaining bents were replaced by framed timber bents on 8-ft centers resting on concrete footings.



Winona Flume and Timber Arch

Length, 210 Ft; Height, 33 Ft; Capacity, 150 Cu Ft per Sec

The arch contains 8,660 fbm of timber and is supported on four heavy concrete and field-stone pedestals, carried down about 10 ft into the earth for safety against heaving from frost action as well as to take the thrust of the arch. Iron connections support the timber on the concrete footings. The timber arch, which has proved to be a safe and economical structure, cost slightly less per linear foot than the trestle bents. Complete with footings, its cost was about $23 per lin ft. The arch was built about an inch above grade to allow for some settlement under load. However, to date no settlement has occurred.

During years of contraction and expansion from extreme temperature changes and of heaving of the pile bents due to frost action, the riveted joints ceased to be water-tight, and leakage threatened to wash out the structure. In this emergency, it was recalled that the old canoe men of Ontario used a mixture of pitch and tallow to make water-tight the seams of their birch bark canoes, and that this same mixture was employed successfully on timber canoes and other boats. The decision was made to try the mixture on the leaky flume. The first application proved a success, as it made the flume completely water-tight.

Ordinary hard coal-tar pitch was melted in pots, and to it from 5 to 10 per cent of beef tallow was added. The mixture was brushed inside the flume to form a thin, soft, rubber-like coating at about 60 F. More tallow would have made the mixture softer. It is quite possible that pine tar pitch would be still better for this purpose, but it is not available in the market. For beef tallow we buy the unrefined tallow from a local butcher, just as it comes from the animal. If the tallow is rancid it makes no difference except that it is not so pleasant to work with.

Such a coating has been used on a variety of metal flumes where scour has worn off the galvanizing and rust

has started to form, and it has afforded an economical and satisfactory means of protection. At the time of application the metal should be dry and if possible warm, and the pitch and tallow mixture should be boiling hot. Winter temperatures in Canada occasionally drop to 50 deg (F) below zero. Thus if conditions are not perfect when the protective coating is applied, there will be some scaling off in the spring, especially if the coat is too thick or if too little tallow is used. In the case of important structures, such as the Winona Flume, the coating is repaired as required every spring. For inclined metal flumes, where high velocities cause excessive wear, the pitch and tallow is allowed to run down and form a fairly thick layer in the bottom of the flume. This gives a splendid wearing surface where it is most needed. Corrugated culverts are now manufactured with a protective asphaltic coating that is thickened at the bottom in a similar way. On account of its pliability and adhesvie qualities, the pitch and tallow mixture would undoubtedly be useful for protecting concrete surfaces and for waterproofing foundations.

Broken Flume, After Leaks Had Softened Supports and Undermined Bents

V-Shaped Expansion Joints Installed After a Break in the Flume

Early in the morning of June 1, 1928, a bad break occurred in the Winona Flume, which was then carrying about 50 cu ft per sec. This was probably due to a weakened joint where the rivet holes had become elongated. A slight movement of expansion pulled the sheets apart, so that a leak developed. This softened the ground under a pile bent and caused the bent to sag, thus opening the joint to further leaks. On the morning of June 14 repairs were completed and the water turned on again. During the period of repair, heavy rains occurred, so that water users did not suffer.

After this experience, a V-shaped expansion joint was installed near each end of the flume. Each joint was made of two pieces of galvanized sheet iron, riveted and soldered together at the bottom of the "V" and riveted on each side to a section plate of the flume, from which a strip had been cut to accommodate the joint. The "V" is approximately 5 in. deep and at ordinary temperatures about 3 in. wide. These joints have obviated all trouble from expansion and contraction.

During dry years, when there was a heavy demand for water, it was found that the entrance structure did not allow the flume to be filled to capacity. Therefore in 1930 the sides of the flume were raised approximately a foot for the first 15 ft of its length and about 6 in. for the next 16 ft. This was accomplished by attaching a strip of galvanized sheet iron on each side, raising the cross ties, and splicing the posts to correspond. Raising of the sides in this way at the upper end of the flume increased its capacity to about 150 cu ft per sec.

It was thought that these improvements would make the flume last for a long time without further unusual expense. But the increased use of water gradually started a heavy seepage from the earth canal, at the outlet end of the flume, and sliding ground began to push the bents downhill. As a result the structure was again threatened. When temporary expedients, such as bulkheads, had proved insufficient, it was decided in 1931 to drain the sliding ground. A 15-ft trench was excavated from a point below the concrete outlet to the face of the coulee. A timber cut-off wall was installed and backfilled by puddling, and a rock drainage channel was constructed, leading from the front side of the wall and discharging into the coulee. The concrete paving below the outlet was extended about 30 ft downstream, and rock riprap was placed below the concrete after the eroded bottom of the canal had been filled in with clay.

Entrance Conditions for a Flow of 126 Cu Ft per Sec

Capacity of Flume, Increased by Raising Its Sides for First 30 Ft

Along each side of the flume a trench was dug to the bottom of the pile bents, so that heavy braces could be installed from the bottom of each bent to the bottom of the next. The lower end of the lowest brace rested against the heavy block of concrete on which the abutment of the timber arch rests. These two trenches were then filled with rock so that they constitute parallel drains running downhill from the concrete outlet. All the drains carry a little seepage water, and the ground around the concrete outlet and the bents is now stabilized. If the pitch and tallow inner coating is kept intact, the flume will last a long time. About once in five years the timber arch, bents, and such should be given a pressure spray treatment of creosote oil, with special attention to the joints, where rotting first starts.

Inspection of Flume in October Showed No Leakage

The checkered history of the Winona Flume has emphasized a number of useful lessons. Although the last word concerning it cannot yet be written, its record to date is of interest. An inspection of the flume in October 1933, at the end of the irrigation season, showed that not one drop leaked through the battered riveted joints of this old flume. This is a high recommendation for the pitch and tallow lining.

Use of the pitch and tallow coating was first suggested by G. H. Patrick, Canal Superintendent of the Northern Division of the Western Section Irrigation System, and the timber arch was designed by G. P. F. Boese, Assistant Engineer of this system. The Chief Engineer of the Department of Natural Resources, and Superintendent of Operation and Maintenance, Eastern Section, Canadian Pacific Railway Company, is A. Griffin, M. Am. Soc. C.E.

Tunnel Construction–Colorado River Aqueduct

New Methods Developed for Rapid Driving and Muck Removal

By F. E. WEYMOUTH

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

GENERAL MANAGER AND CHIEF ENGINEER, THE METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA, LOS ANGELES

WITH an aggregate of 90 miles of 16-ft tunnel to drive, the engineers of the Colorado River Aqueduct have divided the work between their own forces and those of 13 separate contracting firms. Rivalry is therefore keen to expedite every movement in the cycle of drilling, loading, shooting, ventilating, and mucking so that maximum progress will be made. For example, to eliminate as small a delay as a half minute in placing an empty muck car at the tunnel heading, some ingenious car-switching devices, such as vertical-lift "cherry-pickers" and "grasshoppers," have been developed. Maximum drilling speed is attained by the use of a new type of air drill in which a constant pressure on the steel is maintained automatically. These and many other innovations that have been adopted, are contributing to the rapid completion of this phase of a $210,000,000 undertaking.

CARRIAGE FOR ERECTING TIMBER SETS

NOW that 15 miles of the 16-ft tunnel of the Colorado River Aqueduct has been driven, the excavating program of the Metropolitan Water District of Southern California on this project is approaching maximum rate of progress. In February 1934 an advance of 3.17 miles was made, so that on March 1 the tunnel excavation was 16.5 per cent completed. The District's main aqueduct includes 29 tunnels, totaling 91.34 miles in length. These are located at intervals along the entire length of the main aqueduct, extending from Parker Reservoir on the Colorado River westerly to the Cajalco Reservoir, a total distance of 241.5 miles. The estimated cost of the project, including the distributing system, is $209,420,000.

COLORADO RIVER AQUEDUCT PROJECT

To supplement the insufficient and decreasing yield of local sources of water is the purpose of the Colorado River Aqueduct, which will furnish Los Angeles and its adjacent metropolitan area with a permanent, dependable, and ample supply. In addition there will be 150 miles of distributing mains from the terminal reservoir to the various member cities of the District. At present, work is actively under way on all the tunnels of the main aqueduct. Construction of the Parker Dam will be started soon.

Thirteen cities, including Los Angeles, make up the Metropolitan Water District of Southern California. This coastal area includes 1,400,000 acres of fertile, semi-arid, habitable land. It is intensively developed and has a population of 2,491,000, according to a 1930 estimate.

Parker Reservoir, 150 miles downstream from Boulder Dam, will serve as a diversion for the aqueduct and in addition, at some later date, will make low-cost power available for pumping. The dam will aid diversion by complete removal of all silt. Five pump lifts, aggregating 1,619 ft, will be required because the water surface at the diversion site is at elevation 378 ft, or more than 1,000 ft below that necessary to provide pressure distribution to the coastal cities. Power for pumping will be supplied from Boulder Dam.

INSTALLATION OF UTILITIES COMPLETED

Besides the tunnels, the main aqueduct will embrace 27.2 miles of siphon construction, 55.7 miles of concrete conduit, and 65.01 miles of concrete-lined canal, as shown in Fig. 1. There will be several regulating reservoirs along the line. The aqueduct will have a maximum capacity of 1,605 cu ft per sec. The tunnels and the conduit sections will be about 16 ft in finished diameter, and the open canal sections will be 10.2 ft deep and 20 ft wide at the base, the sides sloping to a width of about 50 ft at the water line. The aqueduct traverses a mountainous and desert region, across which it was necessary to build 148 miles of surfaced highways, 468 miles of high-tension power lines, 1,049 circuit miles of telephone lines, and 180 miles of water supply lines before construction of the main features could be undertaken. These utilities are now completed. Financed by a loan

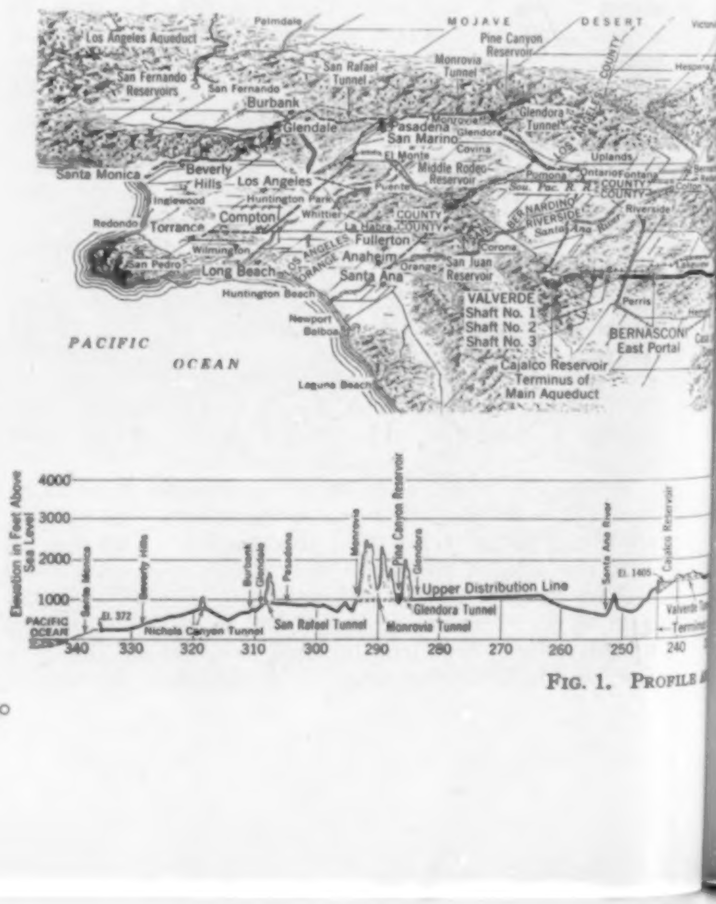


FIG. 1. PROFILE

of $40,000,000 from the Reconstruction Finance Corporation, the District is now employing a total working force of four thousand men.

MATERIALS ENCOUNTERED IN TUNNELING

The longest tunnels on the aqueduct are the East Coachella, 18.27 miles, and the San Jacinto, 12.77 miles. With the exception of the Whitewater tunnels, aggregating 1.93 miles in length, and short sections of the Iron Mountain, Valverde, and Morongo No. 2 tunnels, which are in a sand and gravel formation, all the tunnels are in rock of granitic origin. This rock is characterized by a lack of uniformity; it is broken and seamy, so that supports are generally required. Of the work completed to date, over 60 per cent has been supported either by timber or steel, and an additional 14 per cent has been covered with gunite to prevent spalling and air slacking.

Water flowing at a rate as high as 1,400 gal per min has been encountered in the San Jacinto tunnel, and it is expected that this bore will continue to develop considerable water. The Valverde, Bernasconi, and East Iron Mountain tunnels have small quantities of water, but the others are dry.

With two exceptions all these tunnels will have a finished diameter of 16 ft inside the concrete lining, the outside diameter of the rough excavation being about 18 ft. The exceptions are the Bernasconi and Valverde tunnels, which will have a completed section of 15.25 ft in diameter. The required excavation for the 16-ft sections averages 12.61 cu yd per ft for the timber sections and 10.76 cu yd per ft for the unsupported sections. For the 15.25-ft tunnels, the corresponding measurements are 11.69 cu yd and 9.91 cu yd. The dimensions of the sections are shown in Fig. 2. The work now in progress consists only of excavation; the concrete lining has not yet been started at any point.

Tunnel construction is going forward under the direction of the District and 13 different contracting firms. Contracts have been let for building a total of 57.93 miles of tunnel, and the District is constructing 33.41 miles with its own forces.

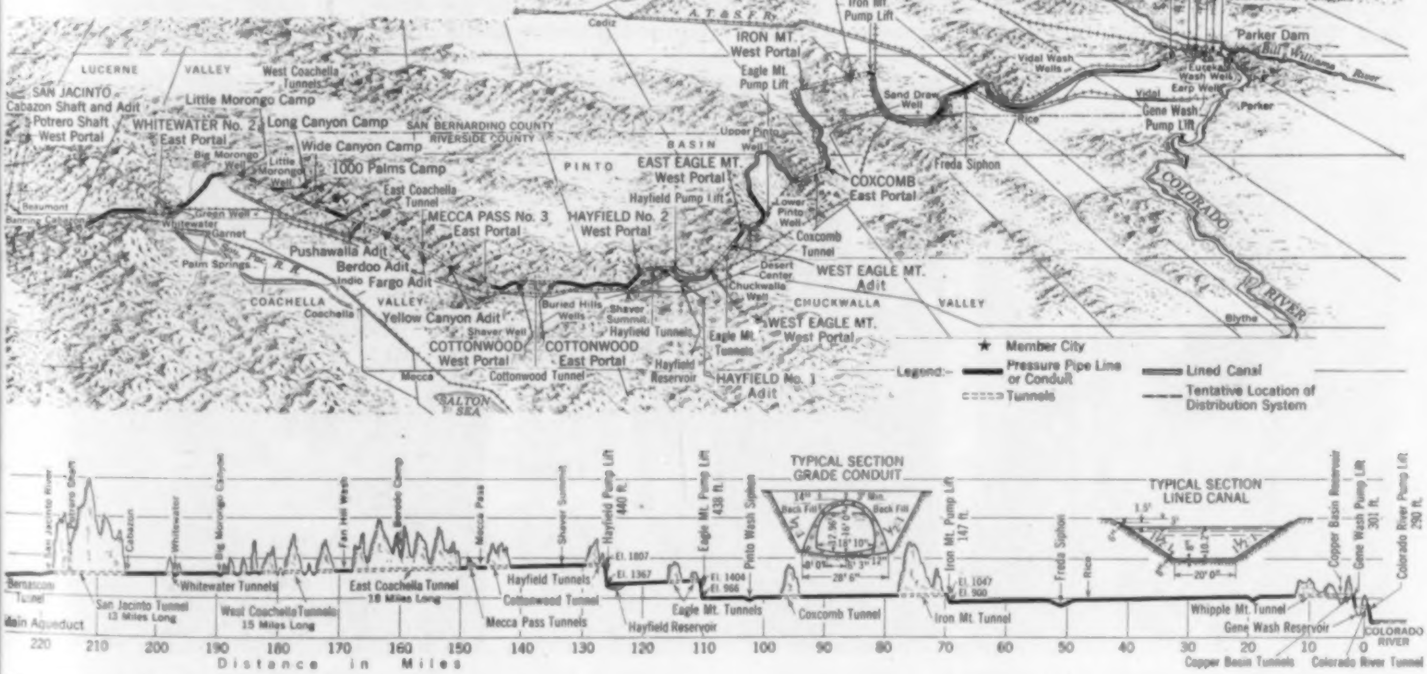
CONSTRUCTION CAMPS FULLY EQUIPPED

Contract work is being pushed from 17 different construction camps, while the District is operating 6 camps for its own work and has 2 more under construction for the Morongo and Blind Canyon tunnels, on which active work has not yet started. The camps are all located adjacent to tunnel or adit portals, or to shafts. They consist of well-ventilated dormitories, mess halls, office buildings, change houses, and the other usual construction buildings.

The type of construction adopted is semi-permanent, usually of wood or a combination of wood and building board. Each camp is lighted by electricity furnished from the District's construction power system. Heating is generally by butane gas or by oil. Each camp is served by telephones from the District's construction telephone system. Many of the camps, including those of the District, are cooled by modern air-conditioning equipment. The kitchen equipment is modern: electrical stoves are used in most cases, and ample refrigeration is provided to keep food safely under desert conditions.

The various camps are built to accommodate forces varying from a maximum of about 180 to a minimum of 40 or 50 men. Each camp is served by a stub road leading from it to the main construction highway built by the District.

Two well-equipped hospitals are being operated in



ALIGNMENT OF COLORADO RIVER AQUEDUCT

these camps: one at Camp Berdoo under the direction of the District, and one at West Eagle Mountain Camp, operated on a joint contract basis for several different contracting firms. Each camp is supplied with a first-aid station, housed in an adequate building and provided with several beds and suitable first-aid equipment, under the direction of a trained nurse.

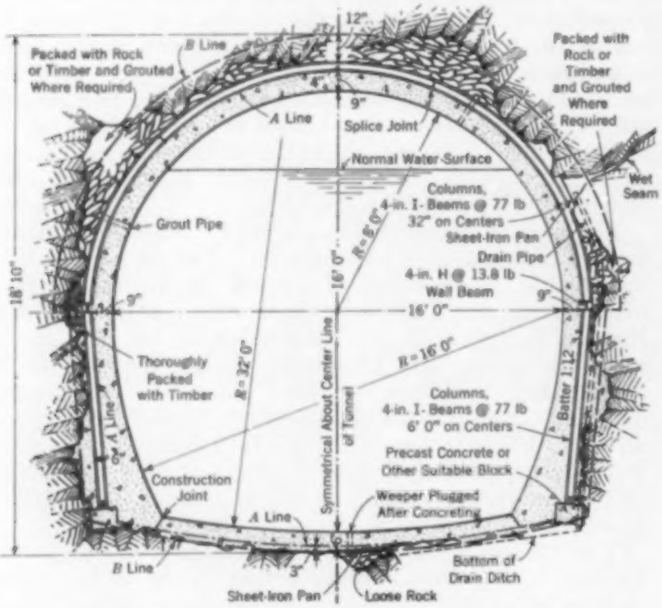


FIG. 2. TYPICAL STEEL-SUPPORTED TUNNEL SECTIONS

PROGRESS EXCELLENT

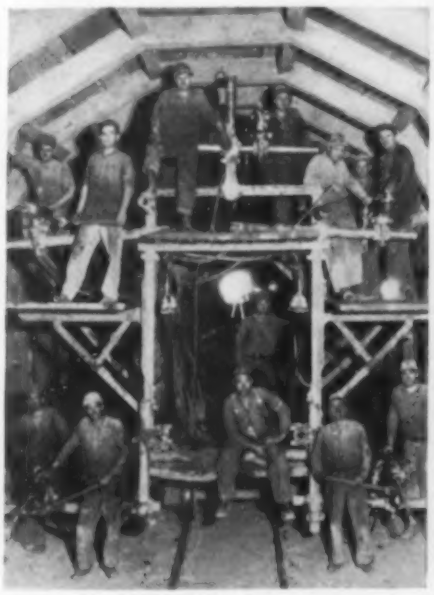
California laws require that no person employed on public works construction shall work more than six days in one week. The various construction organizations meet this requirement either by suspending work on Saturday and Sunday every two weeks or by staggering employment within the crews, so that each man is permitted one day off each week. The former plan is being followed by the majority, including the District, and the result is that the number of shifts worked in a given month is decreased. For this reason any comparison of progress records must be reduced to a shift basis.

For a single heading the best monthly records made to date are 716 ft for rock and 1,027 ft for gravel tunnels. Since the inception of active tunnel driving, monthly progress has gradually increased, totaling 3.17 miles in February for 2,806 working shifts of 8 hr each, an average progress rate of about 6 ft per shift.

The labor forces employed for 3-shift operation usually range from 65 to 85 men per day for each heading. This includes the men working on the outside of the tunnels, as well as those employed underground. With such crews the record of direct labor in recent months has averaged about 31 man-hours per foot of tunnel advance, which is evidence of the efficient crew organization and good tunneling conditions. Additional reasons for the very satisfactory progress being made are the following: (1) the full-face method of drilling; (2) the use of automatic and pneumatic types of rock drills, mounted on special carriages or jumboes; (3) modern mucking and hauling equipment, with ample units provided to cut delays to a minimum; (4) improved car-switching devices, which save time in spotting empty cars at the mucking machines; and (5) the use of large dump cars of 4 to 5-cu yd capacity.

DRILLING AND BLASTING

Machine drills of the pneumatic, automatic-feed type have almost completely replaced the older screw-feed types throughout the work. The new drill is somewhat heavier than the older types, but this is no handicap when a drill carriage is used. The principal advantage of the new drills is that constant pressure can be maintained on the drill bit at all times, permitting maximum speed. Five or six drills are usually mounted on each carriage. From 25 to 60 holes are drilled for full-face blasting, depending on the character of the rock. The holes are arranged in the usual patterns, both hammer and pyramid cuts being made. In softer rock formations, the drilling rounds are from 5 to 8 ft long, but in the firmer formations 10 to 12-ft rounds are drilled. The over-all time required for the drilling operation is from one to three hours. When the harder rock formations permit longer drilling rounds, 50 to 60 holes may be required, and the drilling may take 5 or 6 hr.



Drilling Carriage Mounts Six Drills

Electric Tunnel Mucking Shovel

Rear View of Car-Switching "Grasshopper"

MECHANICAL DEVICES FOR SPEEDING UP TUNNEL PROGRESS COLORADO RIVER AQUEDUCT

Blasting is accomplished with both 25 and 40 per cent gelatine dynamite in $1^1/_4$ by 12-in. cartridges. These are generally wrapped in red instead of the usual yellow paper, so that any unexploded powder may be more easily detected in the muck pile. About $2^1/_2$ lb of powder per cubic yard of rock broken is required under average conditions. Blasts are usually fired electrically from a 440-v circuit used exclusively for this purpose. Firing circuits are protected at all times by two switches which are kept locked in an open position.

VARIETY OF MUCKING EQUIPMENT

Several types of mucking machines are in use on the aqueduct. These may be broadly classified as either the shovel type or the conveyor loader type, the latter predominating. Several muckers have been newly developed on the work which are somewhat similar to the older conveyor type, but which load by a crowding motion of the bucket or boom without any forward movement of the machine as a whole. These latter machines are still in the early stages of development. All the mucking machines in use are operated by electricity.

There are five electrically operated shovels which were especially built for tunneling. These have a bucket capacity of $1^1/_4$ yd, which facilitates the handling of large rocks. With one exception all are mounted on caterpillars. The time required for the mucking operation varies widely, depending on the length of round blasted, the size of the muck, and the type of car-switching device used for serving empty cars to the mucker. From 2 to 3 hr are usually required to completely remove the muck from a full-face round of 6 to 10 ft.

CAR-SWITCHING DEVICES SAVE HALF MINUTES

Mucking machines cannot be kept digging continuously because of the time required to replace a loaded car with an empty one. This lost time usually averages from $^1/_2$ to 2 min for each car, which is approximately equal to the time required to load the car. The tunnel constructors are always seeking to improve switching and mucking devices in order to save this lost time. The "cherry-picker" type of hydraulic or air hoist has been in general use on tunnels for many years. This device is operated on an A-frame support over a single track. It raises the empty car and transfers it to a platform at the side of the track, permitting the loaded train to pass. The empty car is then returned to the main track, and pushed into loading position at the mucker. The operation requires shifting of the main tunnel track from the side to center position as the work advances. In addition to this usual type of "cherry-picker," the other car-switching devices in use are (1) the King "grasshopper," (2) the Dixon conveyor, (3) the California switch, and (4) the vertical-lift "cherry picker." These have been newly developed on aqueduct work.

The King "grasshopper" is a steel-frame device about 150 ft long, so arranged that empty cars are drawn up a ramp from the tunnel track at one end to an upper deck, along which they are moved for lowering to the track at the heading end by means of another ramp. Loaded cars are stored under this deck between the side frames of the machine. Thus an entire train of about eight cars may be drawn up the ramp to the deck of the machine and released to the mucker as needed. The ramps at both ends are hinged for raising when necessary to pass the loaded train along the tunnel track. The front end of this device also serves as a drill carriage.

Somewhat similar in shape and size to the King "grasshopper" is the Dixon conveyor. Its steel frame supports a belt conveyor, on one end of which a mucking machine deposits the excavated material for coneeyance to the empty cars under the other end of the machine. The space under the conveyor, between the side frames of the device, provides storage for loaded cars. While

Pushwalla Camp on the Coachella Division

the mucker is working continuously at one end of the conveyor, an entire train may be loaded at the other end without loss of time. When one train has been filled, the loading operation is delayed while the loaded train is drawn out and the empty one is put in.

Simplicity and inexpensiveness characterize the California switch, a section of double track with frogs and switch points at each end, built as a rigid unit so that it can be superimposed on a single centered tunnel track and moved forward each day or two to keep pace with the advance in the tunnel heading. In passing from the main track to the switch, cars roll upward about 5 in. to the level of the switch on tapered sections of rail lying flat on the main track rails. Being close to the mucking machines at all times, the switch stores the empty cars, which are drawn to the heading as needed by a rope and a small hoist mounted on the mucker. The time required for switching averages less than one minute per car. The principal advantages of the device are low first cost, fast switching, low cost of operation, easy portability, and large savings in main-line track construction.

The vertical-lift "cherry-picker" is an electric hoist that raises the empty cars vertically to such a height as will permit a loaded car to pass underneath. Like the California switch, it operates with a centered tunnel track, and because it can be kept a distance of about one train length from the tunnel face and is easily moved, it reduces the switching time to about $1^1/_2$ min.

In addition to these devices, one of the tunnel shovels employs an ingenious electric jib derrick mounted at the rear of the shovel. In this case the loaded car is "kicked" down the track by the shovel dipper, and the empty car is immediately set on the main track by the derrick, where it is pushed ahead by hand to its position under the dipper. The time required for the transfer is considerably less than one minute.

Tunnel tracks are built to 36-in. gage with 40-lb rails. In some of the tunnels automatic and hand-operated block signals for the protection of traffic have been installed at the intersections of adits and main tunnel and at portals.

Both the battery and the trolley type of electric locomotive are used for hauling, the battery type predominating. In one or two cases combination battery and trolley type locomotives are in service. Where the

trolley type is used without a battery, the trolley line is kept seevral hundred feet away from the heading, and power is supplied to the locomotive while at the heading through an insulated cable wound on an automatic reel. The locomotives vary in size from 4 to 8 tons, the larger sizes being in general use for muck haulage, whereas the smaller ones are employed for switching and inspection purposes. Eight or ten loaded muck cars usually comprise a train.

Side-dump cars of 4 and 5-cu yd capacity are in general use throughout the tunnel. Both Western and mine types are in service. They are ordinarily built with all-steel bodies and are equipped with roller bearings. Automatic couplers are in use on the aqueduct, although the older link-and-pin type still predominates.

COMPRESSED AIR SUPPLY

Compressor plants on the tunnel work generally consist of two or three units at each camp. Each unit comprises a two-stage compressor and an air receiver of about 200-cu ft capacity. The compressors on the District's work are directly driven by synchronous motors, but the major part of the contractor's units are belt driven from such motors. The average displacement capacity of all the compressor plants now in use is 1,500 cu ft per min for a single-face heading and 2,090 cu ft per min for a two-face heading.

These plants operate at about 100-lb pressure, supplying the air for drilling within the tunnel through 6-in. air pipes laid along the invert. Where two headings are worked from a common plant, it is possible to provide a satisfactory supply of compressed air with a somewhat smaller plant than where only one heading is being worked. However, experience on the aqueduct has shown that it is difficult to alternate the drilling routine between the two headings. If delays are to be avoided, drilling must be carried on in each heading independently, a practice which frequently leads to overlapping of drilling periods, necessitating maximum air demand for short intervals.

VENTILATION BY EXHAUSTING AND BLOWING

In order to provide the tunnels with an ample supply of fresh air and to clear away powder gases with the least delay, a minimum of about 3,000 cu ft of air per minute is required at each heading. This is usually supplied through a single pipe from a standard blower installation at the portals or adits. The common practice is to supply air either from two rotary positive-displacement blowers connected in parallel or from two centrifugal compressors mounted in series, arranged so that the air flow can be reversed.

Ventilating pipes vary in size from 20 to 22 in. They are made of 14-gage metal in 30-ft lengths, belled at one end to provide a slip joint. The joints are made airtight by painting their surfaces with asphalt or tar preparations and wrapping with burlap or canvas. The end of the pipe is usually carried to within 80 or 100 ft of the heading. The usual method of operating the blower equipment is to exhaust the air for a period of from 15 to 20 min immediately after blasting. Then the flow is reversed to blow directly toward the tunnel face. The first period, during which the air is drawn out of the tunnel, is usually sufficient to remove powder gases so that work can be resumed at the heading.

As an aid to ventilation, small blowers and short sections of flexible tubing are used in some of the tunnels to create higher air velocities at the tunnel face while the crews are working. In dry tunnels muck piles are thoroughly sprinkled to lay the dust.

Drill steel $1^1/_4$ in. in diameter, of domestic manufacture, is in general use on aqueduct tunnels. The usual practice is to sharpen and temper the steel in shops located at the working portals. These shops are generally equipped with oil-burning furnaces and standard sharpening tools. In addition, several of them are using automatic heat-treating furnaces with pyrometer controls.

A central steel shop, operated by the District, is equipped with one automatic heat-treating furnace, three auxiliary smithing furnaces, and three sharpeners. This shop, in which three shifts work per day, supplies about 1,500 pieces of steel per 24 hr for drilling operations at eight headings.

Of the entire length of tunnel driven to date, 65 per cent has required supports; therefore this feature of the work is most important. Both timber and steel are in common use for the purpose. A combination method employing steel ribs and wood lagging is now being tried out on District work. Steel supports seem to be well adapted for certain types of dry ground and are ideal for tunneling in gravel, where serious caving might result from a fire in which timber supports were destroyed. At present prices, the cost of using timber supports is somewhat less than that of steel, even when allowance is made for the savings in excavation and concrete afforded by the use of steel.

Where timber is employed, the sets are usually framed at the portals by a power saw. Timber is erected at the face from drill carriages, from loaders, from car-switching devices, such as the King "grasshopper," or from specially built timber carriages. A device known as the Kavanagh timber carriage has recently been developed for use in District tunnels. In addition to accelerating framing work, it has the advantage of affording additional safety to the crews. The principal feature of this carriage is a folding mandrel which, when extended for use, leans inward toward the tunnel face and protects workmen by supporting the freshly blasted rock while the standard timber set is being installed. When this carriage is put in general use it is believed that it will materially assist in accident prevention.

SAFETY MEASURES MAINTAINED

A safety engineer and two assistants are employed by the District to supervise safety work on both contract and District operations. Assistance has also been received from the U. S. Bureau of Mines, which has placed an engineer on the aqueduct work to supervise the training of tunnel crews in first-aid methods. Although such training is not compulsory, tunnel crews usually welcome the opportunity to secure it, and frequently the entire personnel completes the course.

The District's safety program includes the maintenance of a completely equipped mine-rescue outfit and the training of crews in its use. The equipment is mounted on a truck and is available for dispatch with trained crews to any of the tunnel work in case of fire. On the force account work, safety statistics are compiled regularly. Each month a flag is awarded to the camp having the best record.

All the work described in this article is under my supervision. The assistant general manager is J. L. Burkholder; the assistant chief engineer, Julian Hinds; the general superintendent, James Munn; and the division superintendent on the District force account work, R. M. Merriman—all Members Am. Soc. C.E. The construction engineer is J. B. Bond, Assoc. M. Am. Soc. C.E., and the chief electrical engineer, J. M. Gaylord.

Discharge Coefficients for Pipe Orifices

Results of Eight Investigations Collated

By Wallace M. Lansford

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In measuring the discharge from certain pipe lines, a convenient piece of apparatus is an orifice in a cap fixed over the end of the pipe. Similarly, a measurement can be made at any point in a pipe line by the insertion of a diaphragm containing the orifice. When the loss of head through the orifice and the diameter of the pipe and orifice are known, an experimentally determined coefficient must be applied in order to calculate the flow. Many experimenters have worked on the problem of determining the proper value of this coefficient for given conditions. By examining the data of a number of these experimenters and reducing them to a common basis, Mr. Lansford is able to compare their results for the value of the coefficient of discharge. In spite of the fact that the conditions under which the various experiments were performed differed widely, he shows that there is remarkable agreement in the results obtained.

For many years the pipe orifice as a device for measuring water has been a subject of experimentation and discussion by engineers. My object in writing this article was to gather together the data of as many of these investigations as possible and to arrive at certain definite values for the coefficient of discharge for different ratios of the diameter of the orifice to the diameter of the pipe. Only concentric pipe orifices measuring water were studied. Most of these orifices were in thin plates placed in the pipe line as diaphragms. The remaining ones were in a cap on the discharge end of the pipe line or were ring nozzles. Although some of the tests on end orifices, notably those by Judd and by Greve, gave slightly higher values for the coefficient of discharge than did those on diaphragm orifices, all values from diaphragm and from end orifices are plotted on the same diagrams, Figs. 9 and 10. The following condensed reviews of the papers studied are illustrated by sketches of the apparatus used by the various experimenters.

Studies of Orifices Reviewed

In his *Manual of the Mechanics of Engineering and of the Construction of Machines*, fourth edition (translated by Eckley Coxe), page 887, Julius Weisbach gives no dimensions for his orifice but calls it a narrow orifice and a diaphragm. No reference is made to the head, the location of the gage connections, or the kind of gage used. He illustrates his orifice by the sketch in Fig. 1.

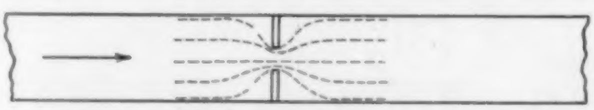

Fig. 1. Weisbach's Orifice

In "Experiments Relating to Hydraulics of Fire Streams," Transactions, Vol. 21, 1889, John R. Freeman reports experiments on ring nozzles. One test was made with a square edged orifice on the end of a $2^{1}/_{2}$-in. pipe. The nozzles were of the type shown in Fig. 2 and were attached to a $2^{1}/_{2}$-in. fire hose from 150 to 300 ft in length. This long channel of approach gave very excellent results. The head on the orifice was measured by means of an open-end glass tube filled with mercury and connected to a piezometer ring.

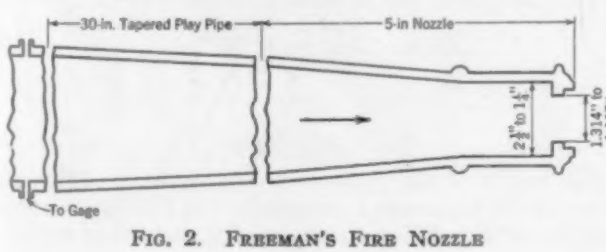

Fig. 2. Freeman's Fire Nozzle

This ring was located at the base of the 30-in. tapered play pipe, approximately 35 in. from the orifice, and was open to the stream the entire distance around the pipe. The pressure head on the nozzle was varied from 20 to 60 lb per sq in.

Tests made with apparatus essentially like that illustrated in Fig. 3 are reported in unpublished theses written for the University of Illinois, as follows: "The Vertical Jet as a Means of Measuring Water," by Stanley Gardner Cutler and Roger Dearborn Marsden, 1908; "The Measurement of Water by Means of Vertical Jets," by Bruce Leroy Jones and Amund Marius Korsmo, 1909; "The Measurement of Water by Means of the Vertical Jet," by Clarence Irwin Haven and Harry Francis Jahn, 1912.

In these three investigations the head on the orifice was found by measuring the height of the jet. This was done by sighting over the top of the jet and reading the head on two level rods, one in front of, and one behind the jet. The measurement obtained was slightly less than the sum of the pressure and velocity heads in the pipe near the orifice, due to the loss through the orifice, and was corrected by increasing the head about 1 per cent. In the experiments, the height of the jet was varied from approximately 1 ft to 4.5 ft.

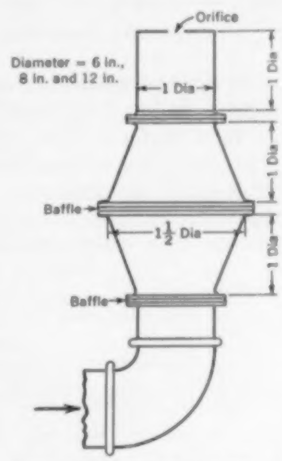

Fig. 3. Method of Testing Vertical Jet Orifice
Diameters of 6, 8, and 10 In. Tested

Another investigation is reported in the article, "The Diaphragm Method of Measuring the Velocity of Fluid Flow in Pipes," by Holbrook Gaskell, Jr., published in the *Minutes of Proceedings of the Institution of Civil Engineers*, Vol. 197 (1914). In Mr. Gaskell's tests the pressure holes, to which pipes leading to a U-tube were connected, were drilled radially through the pipe into the diaphragm, one from each side, as shown in Fig. 4. They terminated in $^{1}/_{4}$-in. holes, drilled into the disk parallel

to the direction of flow, one facing upstream and the other down. In some of the experiments, orifice plates $^1/_{16}$ in.

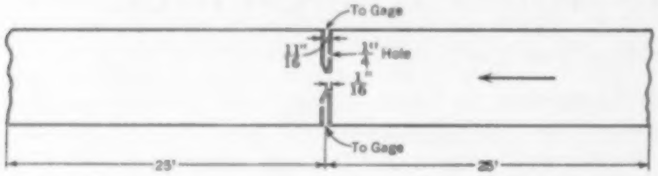

FIG. 4. GASKELL'S DIAPHRAGM

thick were used and the pressure holes were drilled radially into the flanges that held the plate. The head on the orifice was varied from 8 in. to slightly more than 40 in. of water.

In the calculations in his article, "Experiments on Water Flow Through Pipe Orifices," published in the *Transactions of the American Society of Mechanical Engineers*, Vol. 38 (1916), Horace Judd used pressure-head readings made by means of open glass piezometers connected to a pipe at distances of one-half pipe diameter each way from the plane of the diaphragm, and in the case of the cap orifices, at a distance of $7^1/_2$ in. from the plane of the orifice. The pressure head was varied from 0.5 to 8.6 ft. The experimental set-up is shown in Fig. 5.

BOTH DIAPHRAGMS AND CAP ORIFICES USED IN SERIES OF TESTS

In *Bulletin No. 109* (1918) of the University of Illinois Engineering Experiment Station, Raymond E. Davis and Harvey H. Jordan discuss "The Orifice as a Means of Measuring Flow of Water Through a Pipe." They

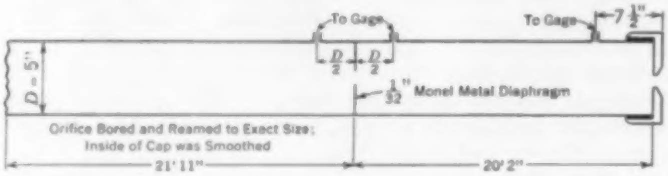

FIG. 5. JUDD'S DIAPHRAGM AND CAP ORIFICE

used air-water and mercury differential gages to measure a drop in head through the orifice ranging from 0.01 to 50 ft of water. They concluded from their experiments that the gage connections to the pipe should be located preferably at distances of 0.8 D upstream and 0.4 D downstream from the orifice, where D equals the diameter of the pipe, there being two pressure openings at dia-

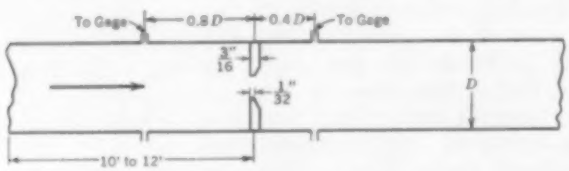

FIG. 6. DIAPHRAGM ORIFICE OF DAVIS AND JORDAN

metrically opposite points in each section. The location of the diaphragm and connections is given in Fig. 6.

Orifice calibrations made by Prof. F. W. Greve at Purdue University for the Layne and Bowler Company are described in the pamphlet, *Measurement of Water Flow Through Pipe Orifice with Free Discharge*, published by Layne and Bowler, Inc., Memphis, Tenn., in 1927. Professor Greve used an open piezometer to measure the pressure head in a pipe at a section 2 ft from the orifice, as shown in Fig. 7. Hose connections to the gage were made from $^1/_4$-in. tapped holes, spaced at the top, bottom, and sides of the pipe. The pressure head was varied from about 0.4 to 5.0 ft.

An investigation by S. R. Beitler and Paul Bucher is reported in the *Transactions of the American Society of Mechanical Engineers*, Vol. 52 (1930), in the article, "The Flow of Fluids Through Orifices in Six-Inch Pipes." In these tests multiple air-water and mercury differential gages were used to indicate the head on the orifice. They were connected to the pipe as indicated in Fig. 8. The one located downstream at the vena contracta measured the drop in pressure through the orifice. The location of this section varied from 0.8 D, when $\frac{D_o}{D}$ equals 0.1477, to 0.2 D, when $\frac{D_o}{D}$ equals 0.8391, where D_o is the

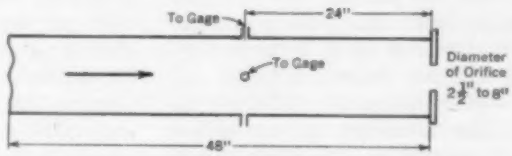

FIG. 7. GREVE'S CAP ORIFICE

diameter of the orifice. The drop in head ranged from slightly less than 2 ft to more than 9 ft.

COMPARISON OF COEFFICIENTS

In order to compare the results secured by the various experimenters, all the coefficients were reduced to the same basis and the following nomenclature was adopted:

h = static head on orifice, in feet

V = average velocity of water in pipe, in feet per second

g = acceleration due to gravity, in feet per second per second

A and A_o = area of pipe and of orifice respectively, in square feet

D and D_o = diameter of pipe and of orifice respectively, in feet

Q = discharge, in cubic feet per second

c_1, c_2, and c_3 = coefficients of discharge, defined in various ways

Then

$$Q = c_1 A_o \sqrt{2g\left(h + \frac{V^2}{2g}\right)} = c_1 A_o \sqrt{2g\left(h + \frac{Q^2}{A^2\,2g}\right)}$$

$$\text{from which } Q = \frac{c_1 A A_o}{\sqrt{A^2 - A_o^2 c_1^2}}\sqrt{2gh}$$

$$= \frac{c_1 A_o}{\sqrt{1 - c_1^2\left(\frac{D_o}{D}\right)^4}}\sqrt{2gh} \quad . \; . \; . \; . \; [1]$$

or

$$Q = \frac{c_2 A A_o}{\sqrt{A^2 - A_o^2}}\sqrt{2gh} \quad . \; . \; . \; . \; . \; . \; [2]$$

or

$$Q = c_3 A_o \sqrt{2gh} \quad . \; . \; . \; . \; . \; . \; . \; . \; [3]$$

The results of the various experimenters, as calculated by means of Equation 1, are plotted in Fig. 9. A simple transformation of Equations 1 and 2 was found useful in

recalculating the data of Judd and that of Beitler and Bucher, as follows:

$$c_1 = \frac{c_1 A A_o}{\sqrt{A^2 - A_o^2 c_1^2}} \sqrt{2gh} = \frac{c_2 A A_o}{\sqrt{A^2 - A_o^2}} \sqrt{2gh}$$

Equating and simplifying,

$$c_1 = \frac{c_2}{\sqrt{1 - (1 - c_2^2)\left(\frac{D_o}{D}\right)^4}} \quad . \; . \; . \; . \; . \; [4]$$

In his experiments Professor Greve used a formula which, expressed in the standard terminology of the Society, is:

$$Q = c_4 A_o \sqrt{2gh}$$

in which Q is in gallons per minute; A_o is in square inches; h is in inches; and c_4 is the coefficient of discharge.

Therefore

$$c_1 \frac{AA_o}{\sqrt{A^2 - c_1^2 A_o^2}} \sqrt{2gh} = \frac{c_4 A_o \sqrt{2gh}}{450} = Q \text{ (in cu ft per sec)}$$

Simplifying,

$$c_1 = \frac{1.11c_4}{\sqrt{1 + (1.11c_4)^2\left(\frac{D_o}{D}\right)^4}} \quad . \; . \; . \; . \; . \; [5]$$

This expression was used in determining c_1 from the data of the tests made by Professor Greve.

In Fig. 9 three curves have been drawn, representing the highest, lowest, and probable average values of the coefficient c_1 for various values of $\frac{D_o}{D}$. The values in Table I were taken from the average curve, shown as a heavy line.

In many cases Equation 3 is the most convenient to use in determining the value of Q, especially for a given orifice when it can be reduced to the expression, $Q = c_5 h^{1/2}$, in which $c_5 = c_3 A_o$. In this equation the required

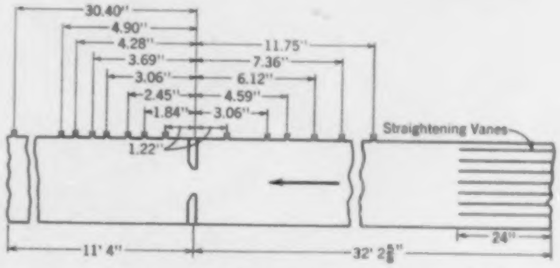

FIG. 8. DIAPHRAGM ORIFICE OF BEITLER AND BUCHER

values of c_3 are obtained by making the following transformations:

$$c_3 = \frac{c_2}{\sqrt{1 - \left(\frac{D_o}{D}\right)^4}} \text{ and } c_3 = \frac{c_1}{\sqrt{1 - c_1^2\left(\frac{D_o}{D}\right)^4}}$$

These values are plotted with the corresponding values of $\frac{D_o}{D}$ in Fig. 10. The values in Table II are taken from the curve in this figure.

TABLE I. AVERAGE VALUES OF c_1 IN EQUATION 1 FOR VARIOUS VALUES OF D_o/D

$\frac{D_o}{D}$	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
c_1	0.612	0.602	0.603	0.610	0.620	0.635	0.658	0.706	0.790	1.00

It is thought that this review of these tests and test results, together with the composite curves in Figs. 9 and 10, may be helpful to engineers and others interested in a simple and inexpensive but accurate means of measuring the flow of water in pipes, such as the pipe orifice affords.

TABLE II. AVERAGE VALUES OF c_3, AS FOUND BY EQUATION 3 FOR VARIOUS VALUES OF D_o/D

$\frac{D_o}{D}$	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
c_3	0.61	0.60	0.61	0.62	0.64	0.66	0.69	0.78	1.07	∞

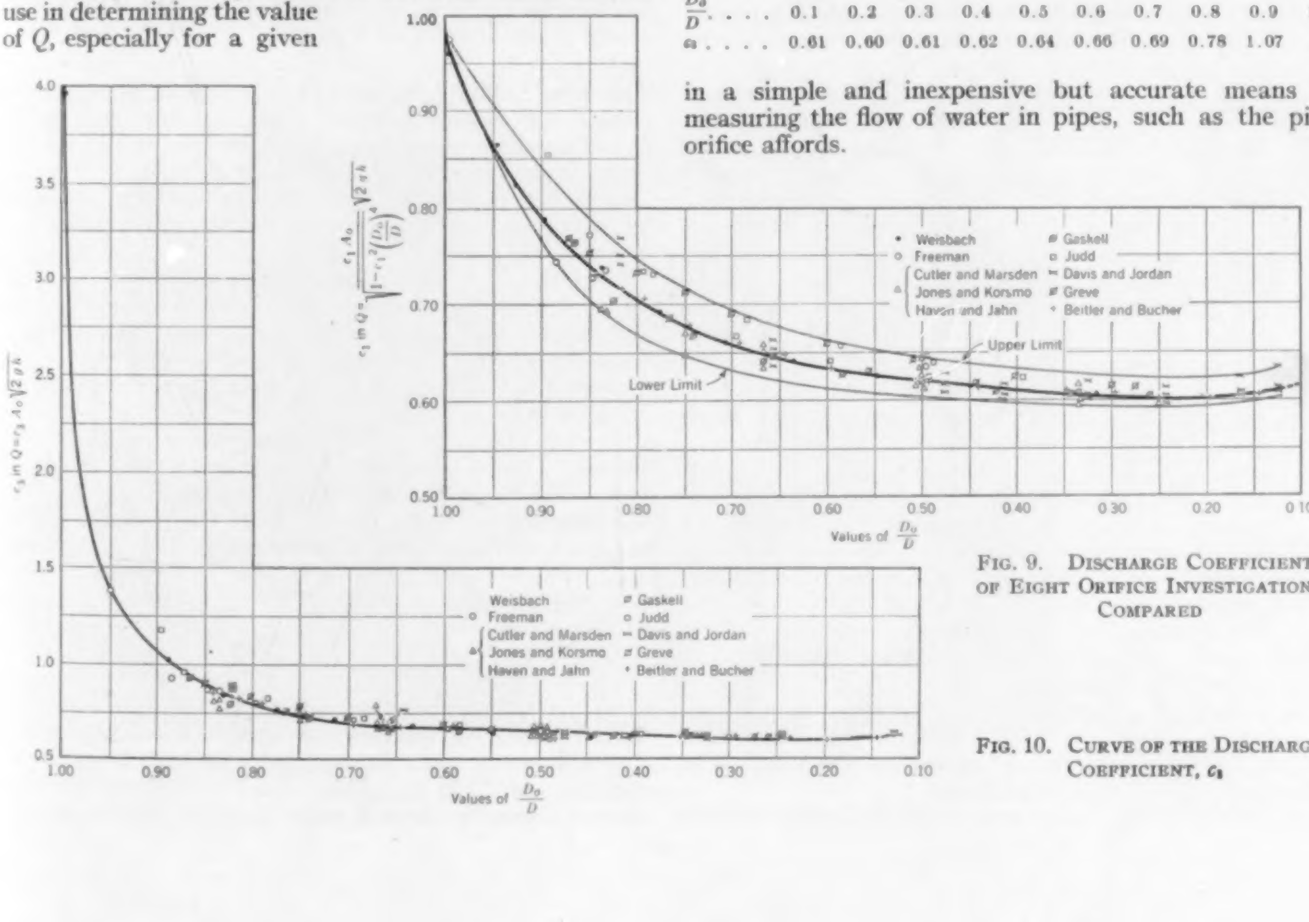

FIG. 9. DISCHARGE COEFFICIENTS OF EIGHT ORIFICE INVESTIGATIONS COMPARED

FIG. 10. CURVE OF THE DISCHARGE COEFFICIENT, c_3

UPSTREAM FACE OF PRIEST DAM PAVED WITH HAND-PLACED ROCK
Tower Contains Control Valves for Auxiliary By-Pass Tunnel

Priest Dam on the Hetch Hetchy Aqueduct

Construction and Maintenance Experience with a Rock and Hydraulic Fill Structure

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An unusual feature of the Priest Dam is its articulated concrete core wall grouted into the rock foundation. It was expected that during the construction and subsequent consolidation of the rock and earth fill of the dam, unbalanced pressures on the core wall would displace it slightly. To avoid rupture resulting from such displacement, the wall was divided into panels vertically by deflection joints of the ball-and-socket type, spaced 50 ft on centers, and horizontally by contraction joints 16 ft apart. All joints were sealed with copper strips. Since completion, the loose rock fill downstream from the core wall has settled 9 ft; the earth on the upstream side, placed by the hydraulic method, has settled 2 1/2 ft; and the core wall has deflected downstream a maximum of 2 1/2 ft. For the past ten years the leakage through the dam has been measured by weirs and has been found to be nearly constant at 1/3 cu ft per sec.

TO provide a forebay reservoir for regulating the flow of water to the Moccasin Power House, the Priest Dam on the Hetch Hetchy Aqueduct, San Francisco water supply project, was built in 1921–1922 at the terminus of the 19-mile Mountain Division tunnel. At normal high water, that is, at elevation 2,240 ft, the Priest Reservoir covers 52 acres and has a capacity of 2,350 acre-ft, or approximately a two days' flow through the aqueduct. The dam is built across Rattlesnake Creek one-half mile south of Priest Station on the Big Oak Flat Road in Tuolumne County, California. At a point about one mile upstream from the dam, the local drainage flow into the creek is diverted and by-passed through a tunnel to the west so as not to contaminate the reservoir. An extensive protective area surrounding the reservoir is owned by the city. The main watershed area is uninhabited.

As shown in Fig. 1, the dam is an earth and rock fill structure, 1,160 ft long and 147 1/2 ft high at its maximum section. It has an articulated concrete core wall and a heavy riprap protection on the upstream face. At the crest, at an elevation of 2,245 ft, the dam is 20 ft wide, and at the base it is 660 ft wide. The dam contains 717,300 cu yd of fill and the core wall, 17,040 cu yd of concrete.

Prior to construction a number of test pits were sunk along the axis of the dam, and tunnels were driven into the abutments. These disclosed a foundation of amphibolite schist, a solid, greenish-gray igneous rock. The overlying soil and weathered material were removed from the core section.

During the early period of excavation at the Priest Portal of the main aqueduct tunnel, 102,700 cu yd of rock spoil from the tunnel was dumped to form the downstream toe of the dam. When the construction of the core wall interfered with this operation, the tunnel spoil, amounting to 1,760 cu yd, was dumped at the upstream toe, where it provided a storage pool for use in placing the hydraulic fill.

STREAM DIVERTED THROUGH A TUNNEL

During the construction of the dam the stream was diverted through an auxiliary outlet tunnel 973 ft long and 6 ft in diameter when finished, with the inlet invert at an elevation of 2,120 ft. This tunnel was driven through the solid foundation rock under the east abutment, and

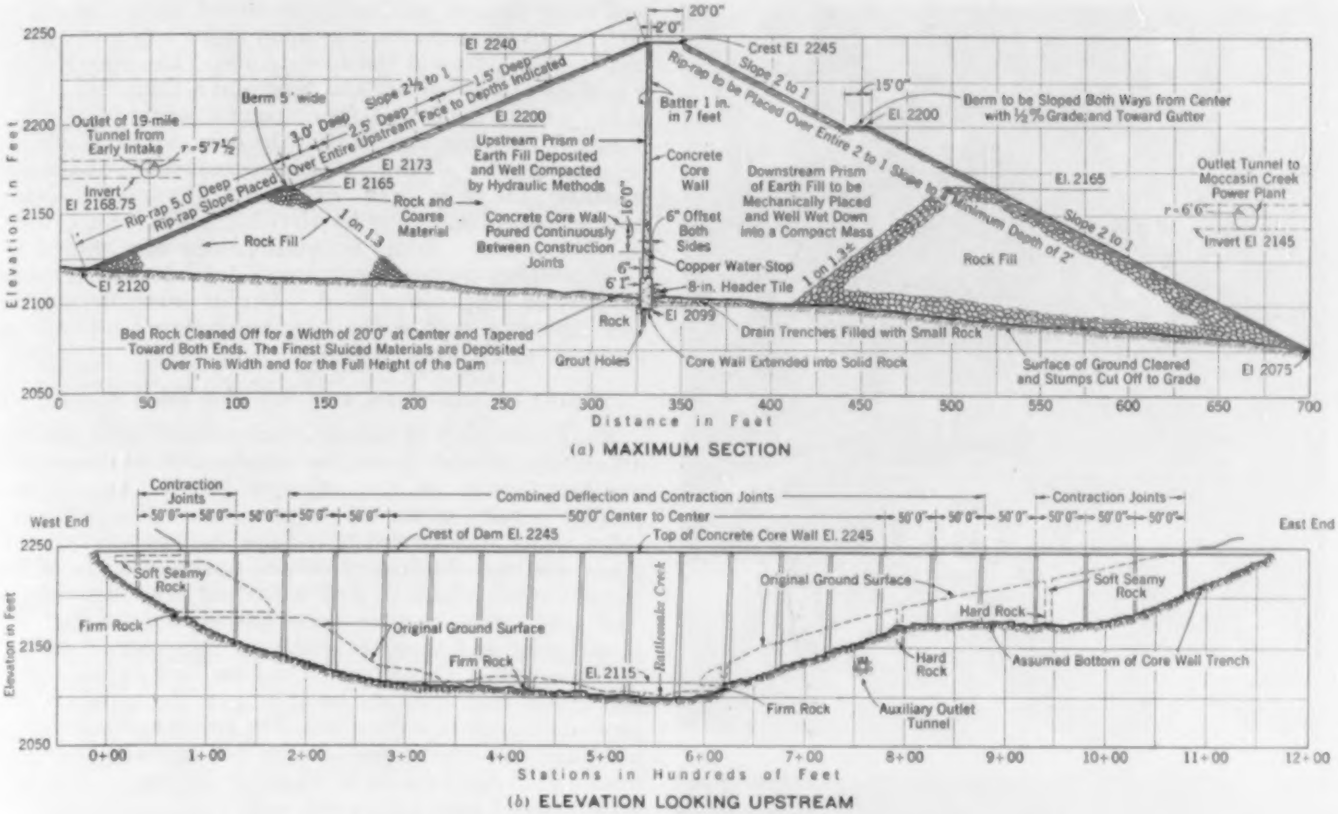
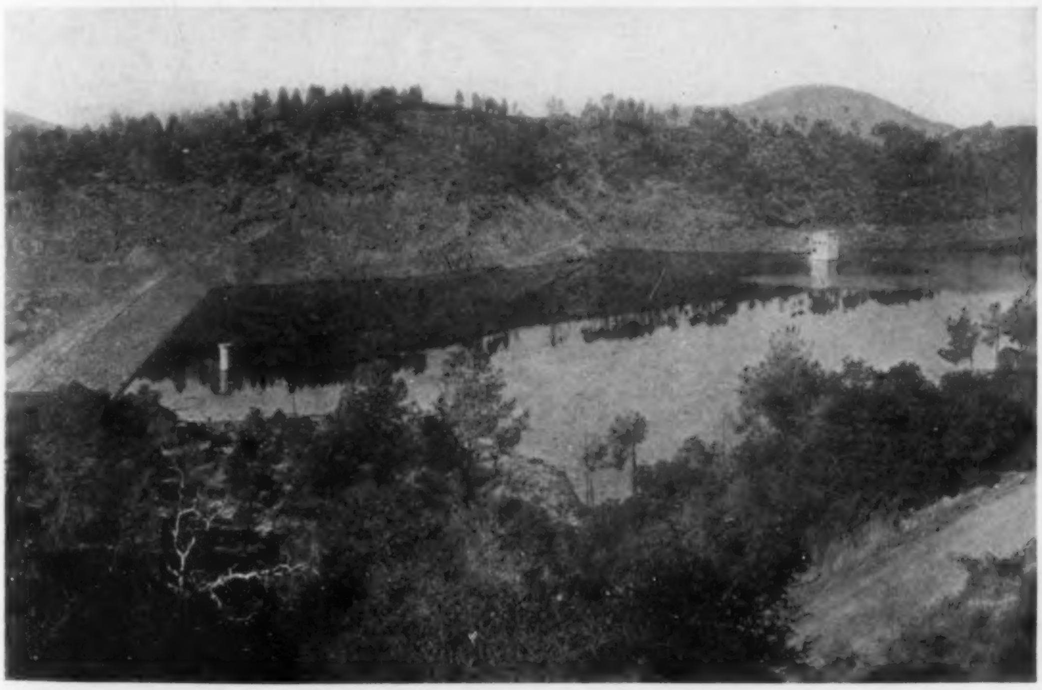


FIG. 1. MAXIMUM SECTION AND DOWNSTREAM ELEVATION OF PRIEST DAM

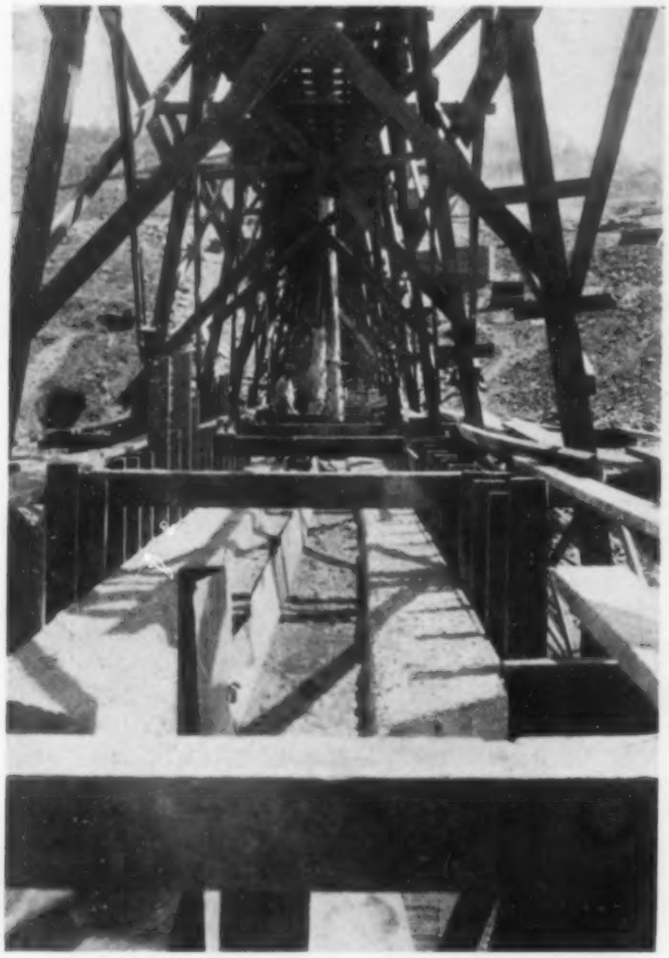
before the construction of the dam, was lined with 12 in. of concrete. Flow through it is controlled by two pairs of 30-in. valves, each pair in tandem. The operating mechanism for the valves is in an enlarged chamber at the base of a cylindrical reinforced concrete tower. This rises from the tunnel near the upstream toe of the dam and has an inside diameter varying from $7^1/_2$ to 6 ft. The tunnel discharges through a concrete lined canal and culvert into Rattlesnake Creek below the dam. The auxiliary outlet may be used to sluice accumulated silt from the reservoir and to drain that part of the reservoir below the main outlet, which is at an elevation of 2,145 ft. As yet, no contingency has arisen that required its use.

A concrete lined overflow spillway with a capacity of 800 cu ft per sec was constructed at the east abutment by excavating and sluicing 7,070 cu yd of rock. The spillway is 350 ft long and 20 ft wide, and has side slopes of one vertical to one-half horizontal. The lip is at an elevation of 2,240 ft.

The main outlet from the reservoir to the power plant is a horseshoe shaped tunnel, 5,370 ft long and 13 ft in clear diameter, lined with 12 in. of concrete. It connects with a surge shaft and thence through penstock pipes with the Moccasin Power House, about two miles to the west of the Priest Dam. The invert inlet of the power tunnel is at an elevation of 2,145 ft. A concrete control tower



PRIEST RESERVOIR, WITH A CAPACITY OF 2,350 ACRE-FT, OR TWO DAYS' FLOW OF THE HETCH HETCHY AQUEDUCT
Square Tower Is at Entrance of Diversion Tunnel to Moccasin Power Plant



POURING THE ARTICULATED CONCRETE CORE WALL OF THE PRIEST DAM

Flexible Construction Made Water-Tight with Copper Sealing Strips, of Which 27 Tons Were Used

with six electrically operated sluice gates, 6 by 8 ft, is provided in the reservoir for regulating the flow of water to the tunnel.

Open-cut excavation for the concrete core wall was carried well into bedrock, the maximum depth of trench being 15 ft. Grout holes were drilled on 6-ft centers, and when the core wall had been carried up to a height of 20 ft, grout was injected into them under a pressure of 75 lb per sq in., to seal the foundation. The core wall is 6 ft thick at the bottom and tapers to a thickness of 2 ft at the crest of the dam. Its maximum height is 160 ft.

Since it was anticipated that the dam would be subject to considerable settlement and occasional unbalanced pressure, the core wall was made flexible by being built in panels 50 ft long by 16 ft high, which were dovetailed together. Joints between panels were sealed by 16-gage sheet-copper water stops, for which a total of 27 tons of copper was used. During construction the core wall was kept vertical by maintaining the fill on each side at the same height.

PART OF UPSTREAM FILL SLUICED INTO PLACE

The greater part of the upstream embankment was hydraulically sluiced from the overburden of loam and weathered rock on the adjacent hillside into flumes roughly parallel to the axis of the dam. The sediment-laden water was carried in a flume laid on a 5 per cent grade and was discharged on the upstream edge of the embankment, where it took a natural slope toward the core wall. The coarser materials settled on the upstream edge and the finer materials were carried toward the core wall. This material was so hard an hour after deposition that walking on it left no footprints. For protection against wave action, the entire upstream slope was paved with hand-placed rock to a thickness tapering from 5 ft at the bottom to 18 in. at the top. This rock was obtained from the power tunnel.

A different method of construction was used for the downstream part of the dam. The space between the rock fill at the toe of the dam and the core wall, and thence up to the crest, was filled with material excavated from the hillside, loaded into cars by steam shovel, and dumped from wooden trestles. The top 20 ft on the upstream side was made by the same method as the bulk of the downstream fill, by dumping from a track laid on the top of the core wall. The material for dump fill was largely loose rock with some earth. After it had been dumped, a hose jet was turned on it to wash the fines toward the core wall, but it was not rolled or otherwise compacted.

For the hydraulic fill a 500-hp motor was used to operate a 12-in. centrifugal pump rated at 4,000 gpm at a 350-ft head. The water was pumped through a 12-in.

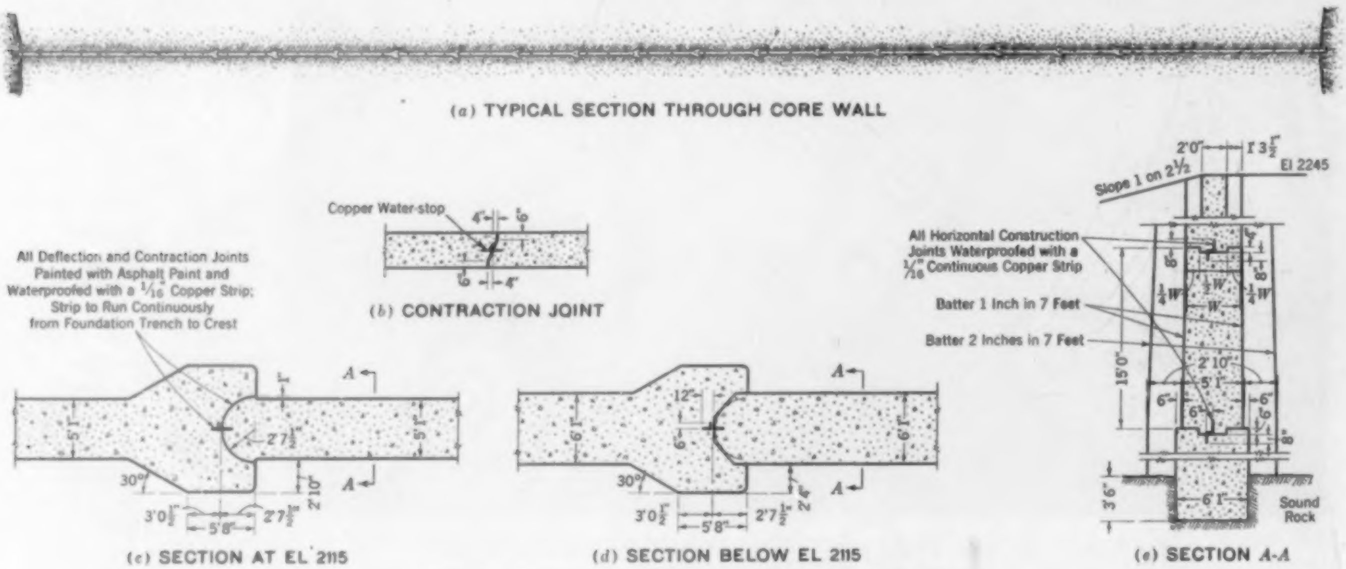


FIG. 2. DETAILS OF ARTICULATED CORE WALL

steel pipe and a 14-in. woodstave pipe to two hydraulic monitors with a 9-in. inlet and 3- to 4-in. nozzle tips. The sediment-laden water reached the dam through 2,500 ft of 20-in. steel flume having 40 side gates for discharge into lateral wood flumes. On the downstream side the dry earth and rock dumped from cars was jetted against the core wall by a hydraulic monitor with a 7-in. inlet and $1^1/_2$ to $2^1/_2$-in. nozzle tips, receiving its water from

TABLE I. PRIEST DAM—PRINCIPAL QUANTITIES AND UNIT COSTS

Item	Quantity in Cu Yd	Unit Cost per Cu Yd
Upstream earth fill	85,616	$0.63
Upstream hydraulic fill	247,656	0.53
Downstream rock fill (tunnel dump)	104,476	....*
Downstream earth fill	264,981	0.63
Riprap on upstream face	14,554	2.00
Concrete core wall	17,043	23.90
Total volume of dam, including core wall	734,326 cu yd	
Total cost of dam and reservoir, including auxiliary outlet tower and tunnel, spillway, concrete core wall, earth and hydraulic fills	$1,062,565.10	

* The rock fill toes, downstream and upstream, were made from material excavated from the tunnel and were charged to the cost of constructing the tunnel.

two 5-in. centrifugal pumps with a capacity of 600 gpm at a 250-ft head, direct connected to a 60-hp motor. The gravity dump part of the dam was excavated by steam shovel, hauled to the dam on tracks of 3-ft gage, and dumped from timber trestles. General data on quantities and unit costs are given in Table I.

It was anticipated that the downstream section of the dam would show great settlement. Actually, after three years it had settled about 6 ft below the top of the core wall. Three years ago material similar to that originally used was excavated from borrow pits and dumped on the downstream part of the dam until the crest had been brought 6 ft above the top of the core wall. The settlement continued so that at the present time the crest is about 3 ft above the top of the core wall. This indicates a total settlement of about 6.2 per cent of the total height. The upstream part of the fill, placed by hydraulic methods, has settled $2^1/_3$ ft, or about 1.7 per cent of the total height. The core wall has bulged downstream to form a very flat curve with a maximum deflection of about $2^1/_2$ ft at the point of greatest height of the dam.

LEAKAGE TO DATE

Weirs were established below the dam to measure the leakage, which amounts to 219,600 gal per 24 hr, equivalent to 0.34 cu ft per sec. This leakage has remained practically constant during the ten years that have passed since the dam was completed, except for a slight variation due to fluctuations in the height of the water in the reservoir. It does not cause any anxiety to the engineers who designed and built the dam.

As the rainfall at the dam is not great—only about 25 in. per year—it is difficult to get a growth of sod on the slopes. Experiments with various native plants have had but indifferent success. Except for esthetic reasons no great importance is attached to this problem.

This dam was constructed in 1921 and 1922 in 18 months by day labor under my direction, as city engineer of San Francisco. It cost approximately one million dollars, including the auxiliary outlet tunnel.

PRIEST DAM DURING CONSTRUCTION, IN 1922, ACROSS RATTLESNAKE CREEK, TUOLUMNE COUNTY, CALIFORNIA
Earth and Rock Fill Structure Built to Regulate Flow to Moccasin Creek Power Plant of Hetch Hetchy Aqueduct

Closure of Crevasses in Small Levees

Sand Bags Placed in Water from Light Timber Trestles Form Effective Barriers

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Without the use of power equipment or highway transport, five crevasses in levees on the St. Francis River were successfully closed while the water from the flood of January 1933 was still running through them. The method here described by Mr. Ross of building a sack dam around or across the break is a usual one for the quick closing of small openings. According to him the same general plan has possibilities for successful application to larger crevasses where the water is flowing through at depths up to 10 ft and at velocities up to 15 ft per sec.

In the flood of January 1933 five crevasses occurred in small levees on the St. Francis River, where it forms the boundary between southeast Missouri and northeast Arkansas. These levees, built to protect adjacent farm land from inundation, varied in height from 6 to 12 ft. The crevasses ranged in length from 81 to 285 ft, and at the time of closure the water running through had a depth of from 3 to 5 ft and an estimated velocity of from 5 to 10 ft per sec. All closures were made while flood water was still running through, thereby appreciably reducing the period of inundation of the overflowed lands.

Because of the damages which would result from subsequent rises that might come before the low-water season, the probability that farmers would hesitate to cultivate lands which might be overflowed if the breaks in the levees were not closed, and the additional cost of closure if the crevasses were allowed to lengthen, steps were taken immediately after the passing of the crest to begin the construction of temporary sand-bag dams designed to confine the river for the remainder of the high-water season. The first step was a careful survey of the situation, since an intimate knowledge of the magnitude and nature of the work required is important before any definite plans for the closure of a crevasse are made. Although time is an important element when more acres of fertile land are being flooded hourly and commerce is being held up on important highways, yet in the end nervous haste is likely to result in more delay than the preparation of a well-thought-out plan. No time, however, was lost in starting to fill sand bags and in arranging for the sawing of the necessary lumber.

After the bills of material were prepared, depots for materials were established, and lines of transportation were decided upon between the depots and the job to provide for the movement of labor and materials. Then departments were organized and proper persons selected to be at the head of each. One general supervisor directed all the work in order to ensure proper division of labor and a definite schedule for the various steps, conditions necessary for the rapid completion of the work.

As avenues of approach to the immediate locality of the crevasses, the drainage ditches paralleling the levees were used. Small boats operating in these ditches furnished the means of communication and transportation. Conditions made it impracticable to employ heavy pile-driving equipment or materials not easily transported by man power and small boats. No new or novel methods were used in closing these crevasses. The same aggressive tactics that are used in all phases of high-water fighting along the Mississippi River were employed here.

By reference to Fig. 1 and the other accompanying illustrations, the construction of a sand-bag dam can be readily understood. The object sought is to lessen the energy of the flowing water by degrees in such a manner as to subject the bottom, on which the dam is to rest, to as little scour as possible. After a careful examination of the bottom, the alignment of the structure is selected. The trestle work or skeleton which binds the dam together to a certain extent is primarily a means for placing the sand bags and for keeping those first placed from being carried away by the current. It consists of uprights, stringers, and cross pieces and is built around the crevasse on the line selected, preferably on the river side. The uprights or posts are spaced longitudinally and transversely as conditions demand. In the structures on the St. Francis River the posts were spaced 4 ft longitudinally and 5 ft transversely with respect to the axis of the dam.

Since the scaffolding is mainly a means of constructing

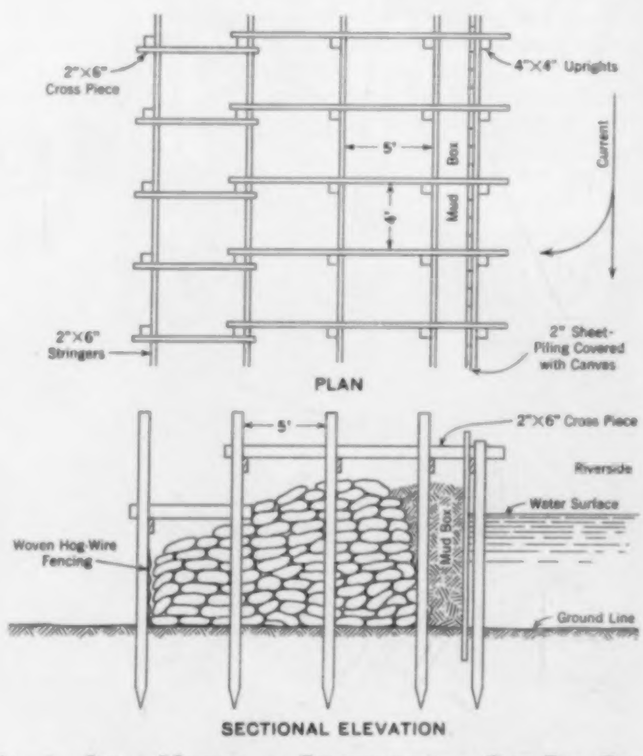

Fig. 1. Ideal Method of Constructing a Sand-Bag Dam
Modification of This Ideal Is Often Required in Practice

the dam, there is no object in having any timbers heavier than necessary to support the planks on which the men walk when carrying sacks. No piece of timber should be heavier than one man can carry. On the St. Francis, the uprights were 4 by 4 in. in cross section and ranged in

LAND SIDE OF COMPLETED SACK DAM IN A ST. FRANCIS RIVER LEVEE, NEAR KENNETT, MO., IN JANUARY 1933

length from 12 to 16 ft. The stringers and cross pieces were 2 by 6 in., of the necessary lengths. The posts were driven to a penetration of at least 4 ft and extended about 3 ft above the proposed height of the finished dam. The distance between the outside rows of posts should be about $2^1/_2$ times the head of water it is expected the dam will be required to withstand.

Construction may be started simultaneously at each end of the crevasse. Post mauls made of wood, weighing from 16 to 25 lb, do very well for driving the uprights. In building the trestle a transverse row of uprights is first driven and a 2 by 6-in. cross piece nailed along their sides at a convenient elevation for runway planks. From a platform of loose planks supported by the cross piece the next row is driven, and the process is repeated until the frame structures started at opposite ends join. Longitudinal 2 by 6-in. stringers are nailed to the successive transverse rows of uprights to stiffen the structure. Woven wire hog fencing $2^1/_2$ to 4 ft in width is then strung along the upstream side of the downstream longitudinal row of uprights below the cross pieces. The lower edge of the wire fencing is forced to the bottom by light stakes driven at frequent intervals. To these the lower edge of the wire is nailed before driving. Extreme care must be taken to make sure that no obstruction keeps the lower edge of the wire off the bottom sufficiently to permit the washing through of sand bags by the current.

SACKS CAREFULLY PLACED

Next comes the work of placing the sand bags. These should not be filled too full—only about three-quarters full to allow for adjustment to inequalities. Thorough tying or sewing of the bags is very essential. Placing of sacks does not consist of dumping them helter-skelter into the water—not by any means. Each sack should be taken from its carrier and lowered into position by men assigned to that task.

It is important that the first sacks be lowered into the water at a point where they will reach the bottom by the time the current carries them against the wire fencing. After the equivalent of several rows of sacks have been allowed to drift against the fencing, the sacks are placed so that they will pave the bottom for 5 or 6 ft back from

TRESTLE CRIB CONSTRUCTION COMPLETED AND READY FOR PLACING OF SACKS

COMPLETED SACK DAM CLOSING THE KENNETT CREVASSE, ST. FRANCIS RIVER, MISSOURI

the fencing, thus lessening the danger of new deep washes being scoured out. Next the sacks are placed in deeper water until the top of the placed sacks is approximately level and the flow of water is evenly distributed over the entire length of the crevasse. From this point up the sacks are placed so that they will be distributed over the entire base of the structure. Their elevation is kept approximately the same longitudinally but they are given a transverse slope of 3 or 4 horizontal to 1 vertical, the river side being kept higher.

After the sacking has reached the top of the water and the free flow is cut off, additional 2 by 6-in. cross pieces are securely nailed to the three upstream uprights in each transverse row, at an elevation slightly above that to which it is proposed to carry the sacking. The cross pieces previously used for supporting the runway are then removed. The stringers between these uprights are also raised, so that no horizontal members are left in the sacking to cause leaks during subsequent rises.

After the sacking has been placed, a mud box is built on the river side of the dam by driving a row of 4 by 4-in. uprights about 3 ft from the dam and parallel to it. These uprights are tied to the dam, and along them a 2 by 6-in. stringer is nailed as a guide for driving 2-in. sheet piling to a penetration of 2 or 3 ft. The downstream side of this sheeting, if not stripped, is then covered with tarpaulins, and the space between the sheeting and the upstream face of the sacking is filled with loose earth.

METHOD GENERALLY APPLICABLE

On the St. Francis River the crevasse closures were accomplished while the water was running through at a fairly shallow depth and low velocity compared with the depth and velocity found in many crevasses on the Mississippi River and elsewhere. However, it is believed that the same general plan, with modifications, might be applied successfully to the closure of crevasses where the water is running through at greater velocities and at twice the depths encountered on the St. Francis River in January 1933.

The general work of both design and construction of the structures referred to was under the direction of Brehon Somervell, M. Am. Soc. C.E., Major, Corps of Engineers, U. S. Army, then District Engineer, Memphis, Tenn.

CAREFUL PLACING OF THE SACKS NECESSARY

Coulomb's Theory of Earth Pressure Rectified

Algebraic and Graphical Methods Used to Determine Point of Application of Resultant

By Pao-Tse Sun

Engineer of Bridges, Kiaochow-Tsinan Railway, Tsingtao, China

Of the classical methods for finding earth pressures on retaining walls, those of Rankine and Coulomb are perhaps best known. Coulomb's theory, here discussed, involves certain inconsistencies. For example it assumes the application of the earth pressure at the third point above the base, contrary to modern experiments. To correct this fallacy, Mr. Sun assumes trapezoidal distribution of pressure on the wall and on the plane of rupture. The solution is relatively simple and has the virtue of agreeing with test values as to the point of application of the earth pressure. In addition, it is claimed to be an improvement, in fully meeting conditions of static equilibrium. As another approach to this old problem, Mr. Sun's method should be of value both to students interested in the theory and to experienced practitioners.

For want of a satisfactory formula, Coulomb's theory of earth pressure is still a tool of the designer. This article constitutes an attempt to remove the one defect in that theory, which seems to have escaped the notice of so many prominent writers on the subject. In Fig. 1 (a), AB is the back of a retaining wall which makes an angle θ with the horizontal; AC is the earth surface, which makes an angle δ with the horizontal; and BC is the plane of rupture, which makes an angle ψ with the horizontal. It is assumed that the wedge of earth, ABC, has the greatest tendency to slide down along BC, so that the angle which the earth resistance, R, on that plane makes with its normal is the angle of internal friction of the earth, assumed to be equal to the angle of repose, ϕ. There are two more forces acting on the wedge: (1) the weight of the earth wedge, W, acting vertically through the center of gravity, G, of the triangle ABC, the thickness of the wedge being unity; and (2) the equilibrant of the earth pressure, E, against the back of the wall, which is assumed to make a constant angle, ω, with its normal. The three forces, W, R, and E, are in equilibrium and therefore form a closed force polygon, as shown in Fig. 1 (b), from which, using w as the unit weight of the earth, the following equation is obtained:

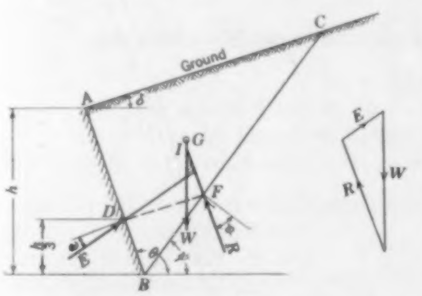

Fig. 1. Forces Acting on a Retaining Wall

$$E = \frac{wh^2}{2} \cdot \frac{\sin(\theta-\delta)\sin(\theta-\psi)\sin(\psi-\phi)}{\sin^2\theta\sin(\psi-\delta)\sin(\theta-\psi+\phi+\omega)} \quad . \; . \; . \; . [1]$$

By differentiating Equation 1 with respect to ψ, equating the result to zero, and substituting the value of ψ thus obtained in Equation 1, the late Prof. Mansfield Merriman obtained E_{max}, the maximum value of E, as follows:

$$E_{max} = \frac{uwh^2}{2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . [2]$$

in which

$$u = \sin^2(\theta-\phi) \div \sin^2\theta\sin(\theta+\omega)\left[1+\sqrt{\frac{\sin(\phi-\delta)\sin(\phi+\omega)}{\sin(\theta-\delta)\sin(\theta+\omega)}}\right]^2 \quad [3]$$

It follows that

$$R = \frac{\sin(\theta+\omega)}{\sin(\psi-\phi)} \cdot \frac{uwh^2}{2} \quad . \; . \; . [4]$$

Much has been written as to what value should be assigned to ω; many hold the view that it is the angle of friction between the earth and the wall, and for lack of reliable data it is usually assumed that ω equals ϕ.

Analyzing the Fallacy

Because the theory did not give definitely the point of application of E, it was invariably assumed that it passes the point at one-third the height from the base of the wall. According to the late George F. Swain, Hon. M. Am. Soc. C.E., it was Winkler who first pointed out the fallacy of such an assumption; for, as can be seen in Fig. 1 (a), when the point D is at $h/3$ from the base, the three forces, W, R, and E, do not meet at one point, so that there remains an unbalanced moment, the magnitude of which is represented by the shaded triangle.

The cause of the fallacy is not far to seek. The method of "the wedge of maximum thrust" is a simple mathematical problem in which there are three given conditions:

(1) $\dfrac{dE}{d\psi} = 0$, and $\dfrac{d^2E}{d\psi^2}$ is negative.

(2) $\Sigma X = 0, \Sigma Y = 0$.

(3) $\Sigma M = 0$.

The first condition enables the angle of rupture, ψ, to be evaluated. The second condition fixes the magnitude of the lateral pressure, E. By means of the third condition, one of the remaining two elements of E ought to be determined, that is, either its direction or its point of application. As the traditional Coulomb's theory assumes E to make a constant angle, ω, with the normal to the back of the wall and also to act at a distance $h/3$ from the base, it is no wonder that the requirement of the nullification of the summation of moments about any point is left unfulfilled.

My studies show that there are two possible ways of rectifying the error, equally consistent with the given conditions of the problem. One is by assumption of triangular distribution, and the other by assumption of trapezoidal distribution of the earth pressures on the two faces of the wedge.

Assumption of Triangular Distribution

In Equations 2 and 4, E and R are seen to vary as the square of h. This fact leads to the off-hand assumption that the earth pressures along the planes AB and BC are

each distributed in the form of a triangle, as shown in Fig. 2. Under this assumption the point of application of E must be at $h/3$ from the base, and therefore is a known quantity. Consequently the angle of obliquity, ω, of E must be taken as an unknown of the problem.

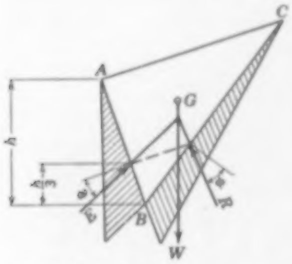

FIG. 2. ASSUMPTION OF TRIANGULAR DISTRIBUTION

Proceeding along this line of attack I have been able to arrive at exactly the same result as by Rankine's theory. This investigation was published in a paper entitled the "Identity of Coulomb's and Rankine's Theories of Earth Pressure," in the *Journal* of the Chinese Institute of Engineers, Vol. 7, No. 3 (September 1932). As Coulomb's theory has always been considered the less objectionable of the two classical theories, the proof of its identity with Rankine's theory is rather discouraging.

The most objectionable feature of Rankine's theory is that according to it an active pressure may have an upward component when the retaining wall leans toward the fill and also when the earth surface slopes away from the top of the wall. This is certainly contrary to common sense as well as to experimental results. In his book, *Design of Walls, Bins, and Grain Elevators,* M. S. Ketchum, M. Am. Soc. C.E., proposed to remedy this by assuming that in all such cases the actual pressure is horizontal and its magnitude cannot be greater than the horizontal component of the theoretical pressure. He called this "Rankine's theory modified," which has been held by the American Railway Engineering Association as standard practice ever since 1917.

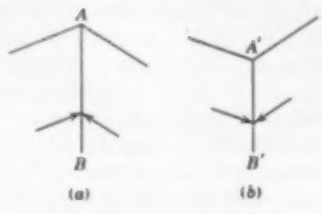

FIG. 3. ACTIVE EARTH PRESSURES

Nothing could be said against such a procedure if it were meant to be merely an empirical rule. But Professor Ketchum based his modification on the following reasoning. If the pressure on a vertical plane must be parallel to the earth surface, then the resulting thrusts acting on the two sides of AB, Fig. 3 (a), would not balance each other. Therefore the maximum active earth pressure cannot have an upward component. This is rather a questionable proof; for one could just as well select the illustration of Fig. 3 (b) and prove that neither can an active pressure have a downward component.

I am of the opinion that just as the free surface of a perfect fluid cannot be inclined, so that of a semi-fluid cannot have such abrupt changes of slope as illustrated in Fig. 3. The sharp corners would perhaps round off into some sort of smooth curves so that the semi-fluid pressure acting on any vertical plane would be parallel to the tangent to the surface at the top of the plane. If this explanation stands, it appears that Rankine's theory is perfectly consistent. The incompatibility between it and experimental results is simply due to the fact that earth is not an ideal semi-fluid and therefore does not follow its laws. All that can be done is to discard the theory. To make unreasonable modification of it is to add inconsistency to what is already inaccurate.

It will be shown that the assumption of trapezoidal distribution of the earth pressures on the planes AB and BC (Fig. 4) is also consistent with Equations 2 and 4. Under this assumption, as in the traditional Coulomb theory, the obliquity ω of the lateral pressure E will be considered as a known constant. Its point of application will be taken as the third unknown.

Since the point of application of R likewise becomes an unknown, a certain assumption will be needed to fix the relative positions of the two points of application. It seems reasonable to assume that the line joining the points of application of E and R is parallel to the earth surface. The adoption of this assumption leads to very simple results.

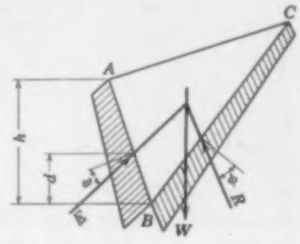

FIG. 4. ASSUMPTION OF TRAPEZOIDAL DISTRIBUTION OF PRESSURE

After the angle of rupture, ψ, has been determined, the point of application of E can be found by the following simple geometric construction. In Fig. 5, AC is the earth surface, AB the back of the wall, BC the plane of rupture, and GW the line of action of the weight, W, of the earth wedge. Draw any line, $D'F'$, parallel to AC. Through point D' draw a line making an angle ω with the normal to AB. Through point F' draw a line making an angle ϕ with the normal to BC. The two lines meet at point I'. Join $I'B$, cutting GW at point I. Draw ID parallel to $I'D'$ and IF parallel to $I'F'$. It can be proved that DF is parallel to AC. Therefore ID is the line of action of E required.

Denote by d the vertical height of the point of application of E from the base of the wall and let

$$d = \frac{vh}{3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad [5]$$

Then the following equation is readily obtained:

$$\begin{aligned} V = \{&\sin(\theta - \psi + \phi + \omega) \\ &\times [\cos\delta \sin(\theta - \psi) + \cos\theta \sin(\psi - \delta)]\} \\ &\div [\sin(\theta - \psi)\sin(\theta + \omega)\cos(\psi - \delta - \phi) \\ &+ \sin(\theta - \psi + \phi + \omega)\cos\theta\sin(\psi - \delta)] \quad . \quad . \quad . [6] \end{aligned}$$

in which ψ is the actual angle of rupture. By means of Equations 2, 3, 5, and 6, the lateral pressure, E, is completely determined.

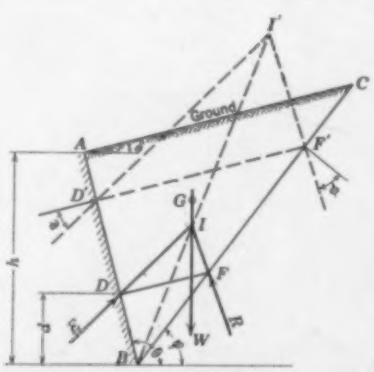

FIG. 5. GRAPHIC DETERMINATION OF POINT OF APPLICATION OF EARTH PRESSURE

It is now possible to find the actual law of distribution of the earth pressure resulting from the foregoing assumptions. Project the trapezoids of pressure of Fig. 4 on the vertical height, h, of the wall, as shown in Fig. 6. Equating the area of the first trapezoid to E/w of Equation 2, the following equation is secured:

$$a + b = uh$$

Equating the height of the center of gravity to d of Equation 5, it is found that

$$\frac{a + 2b}{a + b} = v$$

Then

$$\left.\begin{aligned} a &= (2 - v)uh \\ b &= (v - 1)uh \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \quad [7]$$

Likewise, for the trapezoid of R/w, the following equations are obtained:

$$\left.\begin{aligned} a' &= \frac{\sin (\theta + \omega)}{\sin (\psi - \phi)} (2 - v)uh \\ b' &= \frac{\sin (\theta + \omega)}{\sin (\psi - \phi)} (v - 1)uh \end{aligned}\right\} \quad . \; . \; . \; . \; . \; [8]$$

In both cases the unit pressures at the top and bottom of the wedge vary with h, which is consistent with Equations 2 and 4.

ANALYTICAL METHOD MODIFIED FOR APPLICATION OF REBHANN'S GRAPHICAL SOLUTION

The analytical method is too complicated to be of practical use. Fortunately, by the introduction of a simple modification, the well-known Rebhann's graphical method can be used to solve for the point of application as well as the magnitude of E in conformity with the rectified Coulomb theory here presented.

In Fig. 7 the method is illustrated. Given AB, the back of a retaining wall, and AC, the earth surface; also ω, the angle of friction between the earth and the wall. It is required to find the magnitude and point of application of the earth pressure, E, against the retaining wall. Draw the slope of earth AD, and line BD, making an angle ϕ with the horizontal. Describe a semicircle on BD as the diameter. Draw AE, making an angle $(\phi + \omega)$ with AB. Draw EF perpendicular to BD. With B as center and BF as radius, describe an arc, FH. Draw

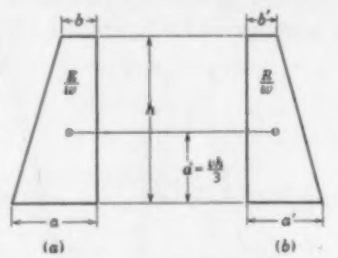

FIG. 6. DEMONSTRATING LAW OF TRAPEZOIDAL DISTRIBUTION

HC parallel to AE. Then CB is the trace of the plane of rupture.

Through C draw a line perpendicular to BD, meeting it at point I, and lay off KI equal to h, the height of the retaining wall. Lay off IL equal to CH. Draw CM parallel to KL, and draw KM. Then the magnitude of E equals the area of the triangle KIM multiplied by the unit weight of the earth. For convenience this triangle is transferred to the left of the figure, as $K'I'M'$. It is to be remembered that under the assumption of trapezoidal distribution, triangle $K'I'M'$ does not represent the true state of pressure distribution on AB.

The foregoing method is essentially that of Rebhann, as demonstrated in Ketchum's treatise. That the triangle KIM is equivalent to the triangle CIL is obvious.

PROPOSED MODIFICATION OF METHOD

As a modification of this procedure, for finding the point of application of E, draw a vertical line, GW, through the center of gravity, G, of the triangle ABC. Through point A draw a line making an angle ω with the normal to AB. Through point C draw a line making an angle ϕ with the normal to BC, meeting the first line at point N. Draw NB, cutting GW at point O. Through point O draw OP parallel to NA. Then OP is the line of action and P the point of application of E required. Likewise, the line OQ drawn parallel to NC is the line of action of R. The trial triangle ANC may be constructed on any line drawn parallel to AC, such as $D'F'$

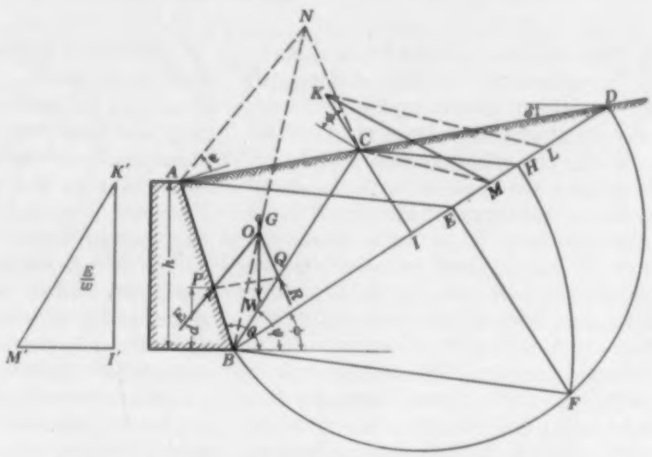

FIG. 7. REBHANN'S GRAPHIC METHOD OF DETERMINING MAGNITUDE AND POINT OF APPLICATION OF EARTH PRESSURE

in Fig. 5. This construction is based on the assumption that PQ (Fig. 7) is always parallel to AC.

Some numerical results obtained by applying this method are given in Table I.

TABLE I. APPLICATION OF RECTIFIED THEORY
Assuming $\phi = \omega = 33°42'$

Assumed Values		Derived Values	
δ	θ	E	d
0°	90°	0.129 wh^2	0.42h
5°	105°	0.221 wh^2	0.36h
5°	90°	0.137 wh^2	0.40h
5°	75°	0.084 wh^2	0.45h

It is significant that in every case the point of application lies between $h/3$ and $h/2$ from the base, which is in general agreement with the results of tests by Steel, Goodrich, and Müller-Breslau.

DESIGN MODIFIED FOR LOADED FILL

In the case of loaded fillings, proceed as follows. Suppose a retaining wall, 15 ft high ($\theta = 105°$) sustains a filling ($\delta = 5°$) which carries a uniform load equivalent to 5 ft of earth. From Table I compute $u = 0.442$ and $v = 1.08$. When $h = 15 + 5 = 20$ ft, by Equation 7, $a = 8.13$ ft and $b = 0.71$ ft. First construct the trapezoid for a height of 20 ft. At 15 ft above the base draw a horizontal line across the figure. Then the lower trapezoid will represent the true pressure distribution on the 15-ft wall required, the top being 2.56 ft wide and the bottom 8.13 ft. Therefore $E = w\left(\frac{2.56 + 8.13}{2}\right) 15 = 80.18\, w$, and by computation it will be found that $d = 6.22$ ft.

The outstanding merits of the rectified theory are: (1) it is consistent with the conditions of static equilibrium, and (2) it agrees more closely with the results of tests as regards the point of application of the lateral pressure.

Flow Around a River Bend Investigated

Tests on Iowa River Support Theory of Helicoidal Flow

By F. L. Blue, Jr., J. K. Herbert, *and* R. L. Lancefield

Second Lieutenants, Corps of Engineers, U. S. Army

Detailed from the Corps of Engineers to the University of Iowa at Iowa City for graduate study in hydraulics, Lieutenants Blue, Herbert, and Lancefield conducted an elaborate investigation on the behavior of the Iowa River in flowing around a sharp bend. The stage of the river was such that it was 125 to 175 ft wide. Not only was the super-elevation of the water at the outside of the bend determined by actual measurement but a theoretical formula based on centrifugal force was developed which gave the super-elevation as close to fact as might be expected. The authors conclude from theoretical considerations and from instrumental observations and measurements on the reach studied that spiral flow exists around a river bend, thus adding weight to a theory which was first advanced by Professor Thomson in England in 1876.

From the time of Dubuat's experiments in the eighteenth century down to the present, investigations have been made to determine the phenomena which accompany the flow of water around bends. The great majority of these investigations have been made on flow in pipes, and because of the complexity of the problem, very little has been discovered regarding flow around river bends. The theory of spiral flow advanced in 1876 by Prof. James Thomson has been accepted almost in its entirety, but without further investigation, by more recent writers on the subject of river hydraulics. The conclusions drawn from some recent work on models discrediting this theory were published in Civil Engineering for May 1933, in the article, "Flow in River Bends," by Herbert D. Vogel, Assoc. M. Am. Soc. C.E., and Paul W. Thompson, Jun. Am. Soc. C.E. However, it is believed that more weight can properly be given to results obtained on an actual river bend.

In the investigation here discussed the purpose was to study the following points: (1) super-elevation of the water surface at the outside bank, (2) presence or absence of spiral flow, (3) longitudinal profile of the water surface at each bank, (4) velocity distribution at various cross sections, and (5) movement of the bend. The bend chosen is at mile 88 on the Iowa River, near Iowa City, Iowa. The very abrupt turn which the stream takes at this point (Fig. 1) makes the flow phenomena unusually susceptible of observation and measurement.

The theory of flow of water around bends is based on the fact that water in motion moves in a straight line unless deflected by the action of some unbalanced force. If water moves in a curve, there must be an unbalanced force in the form of excess pressure tending to push each particle of water toward the inside bank. Since the water has a free surface, such excess pressure can exist only when the water surface is higher at the outside of the bend than at the inside. This difference in elevation was accurately measured in each observation of the bend under investigation.

For any particle of water the required super-elevation is obtained from the equation of centrifugal force,

$$\text{Force} = \frac{\text{Mass} \times \text{Velocity}^2}{\text{Radius of curvature}}$$

By substituting V for velocity, g for the acceleration of gravity, and r for the radius of curvature of the path of the particle, the following equation is obtained:

$$\text{Transverse slope of water surface} = \frac{V^2}{gr}$$

It is difficult to apply this formula in practice because of uncertainty as to the value of the radius of curvature, r. The paths followed differ for different particles, and their average differs both from the line of deepest water and the line halfway between banks.

If certain assumptions are made as to the variation of V or r across the stream, expressions can be obtained for the transverse curve and the super-elevation. These are necessarily approximations, for neither V nor r follows a fixed law in nature. A number of these expressions are given in *Hydraulics of the Miami Flood Control Project*, by S. M. Woodward, M. Am. Soc. C.E., published by the Miami Conservancy District in Dayton, Ohio. A fairly good approximation is obtained by assuming V to be constant at the average velocity, and r constant at the value for the center of the stream. Then,

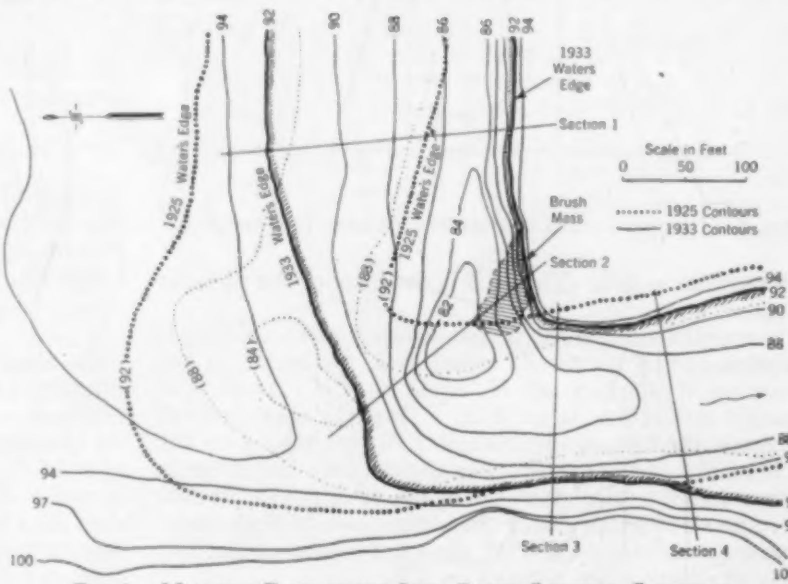

Fig. 1. Map of a Bend in the Iowa River Showing Contours in 1925 and in 1933

$$\text{Difference in elevation of water at the two banks} = \frac{V^2 b}{gr} \quad . \quad . \quad . [1]$$

in which b is the breadth of the stream. This equation

will always give too low a value because the effect of the filaments with the higher velocities more than offsets the effect of the slower filaments, since the velocity enters to the second power in the formula.

If the velocity is zero at each bank, has a maximum value of V_m at the center, and varies between these values according to a parabolic curve, and if the radius of curvature is measured from a fixed point on the bank, then

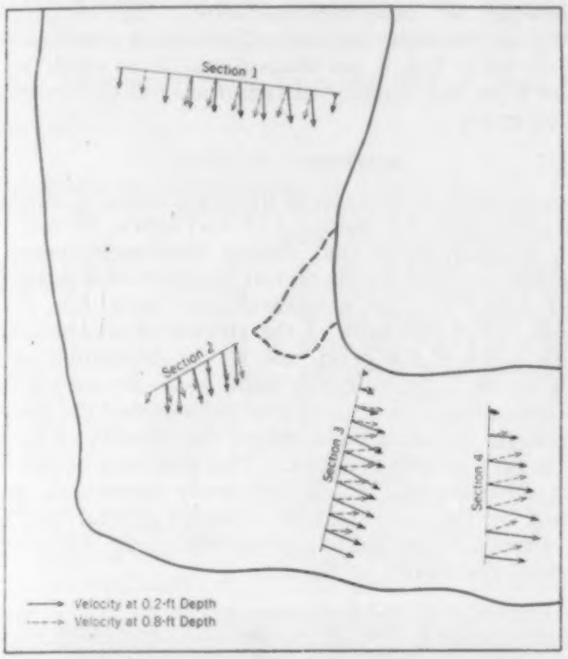

FIG. 2. VELOCITY VECTORS AT 0.2 AND 0.8 OF THE DEPTH, MAY 10, 1933

Total difference in surface elevation =

$$\frac{V_m^2}{g}\left\{\frac{20r}{3b} - \frac{16r^3}{b^3} + \left[\left(\frac{4r^2}{b^2} - 1\right)^2 \log_e\left(\frac{2r+b}{2r-b}\right)\right]\right\} \ldots\ldots [2]$$

in which V_m equals $\frac{3}{2}\,V$, and r equals the radius at the center of the stream.

These equations were applied to the bend in question at Section 3 with the results given in Table I. They show that Equation 1 gives values uniformly low, as was expected, and that Equation 2 gives values closer to the measured differences in elevation. All things considered, the agreement between fact and theory is as close as could be expected.

TABLE I. SUPER-ELEVATION OF WATER AT A BEND IN THE IOWA RIVER

Calculated and Measured Values, in Feet

DATE	BY EQUATION 1	BY EQUATION 2	MEASURED VALUE
April 23, 1933	0.090	0.109	0.125
April 26, 1933	0.080	0.097	0.095
May 3, 1933	0.085	0.103	0.115
May 10, 1933	0.118	0.142	0.200

THEORY OF SPIRAL FLOW INVESTIGATED

The theory of spiral motion of water flowing around bends, first propounded by Professor Thomson, may be explained as follows. If a unit vertical column of water at any point in a bend is imagined, it is apparent that at any depth the difference in pressure on the two sides is the same as the difference at the surface, corresponding to the transverse slope. Since this difference equals $\frac{V^2}{gr}$, in which g is the gravitational constant, $\frac{V^2}{r}$ must be constant for the column. However, the current velocities at the bed of the stream are very much less than those near the surface. Therefore, in order that $\frac{V^2}{r}$ may remain constant as the bottom is approached, r must be reduced. This reduction in the radius of curvature at the bottom as compared to that at the surface results in a movement in which the water at the top flows toward the outside of the bend while that on the bottom flows toward the inside. Near the outside bank the water moves from top to bottom; near the inside bank it moves from bottom to top. A combination of this lateral component and the normal movement of the water parallel to the axis of the stream results in helicoidal flow.

On the bend under investigation, spiral flow was studied by means of a "deflectionometer." This instrument consisted of a vane attached to the lower end of a steel shaft, which was free to rotate inside a pipe. At the top of the pipe a pointer attached to the shaft indicated the direction of the current on a graduated disc. As shown in Fig. 2, spiral flow was definitely found, measurable differences in direction of flow at different depths being detected every time the instrument was used.

Two different directions of spiral movement were noted. Another long bend upstream caused the top layers of the water to move toward the left bank, while the lower layers moved toward the right bank, forming a counterclockwise spiral at Section 1, immediately upstream from the bend under investigation. At Sections 3 and 4, a very definite clockwise spiral was found, the top filaments moving toward the right bank, while on the bottom there was a flow toward the left bank. The existence of these two spirals, although opposite in direction, is in entire agreement with the theory, since the bends generating them are also opposite in direction.

LONGITUDINAL PROFILE AND VELOCITY DISTRIBUTION

As the stream changes its path from straight to curved, the transverse profile of the water surface changes from a horizontal to an inclined line. This change can occur

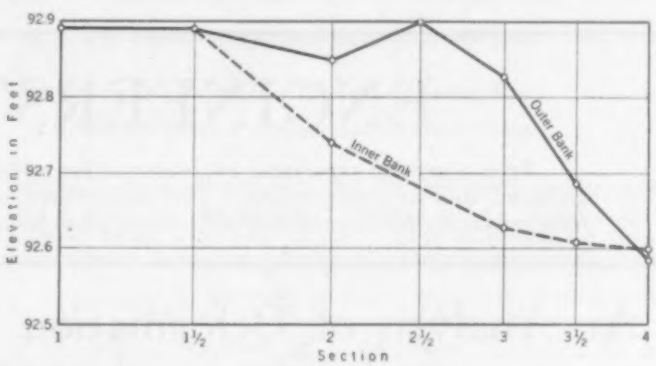

FIG. 3. LONGITUDINAL PROFILES OF WATER SURFACE AROUND BEND, ON MAY 10, 1933

through a drop in the water surface at the inner bank, a rise at the outer bank, or a combination of both. Since there is a natural resistance to any considerable rise at the outer bank, in most bends the required super-elevation is obtained by a drop in the water surface along the inside bank while the water at the outer bank runs along almost level. In very abrupt bends, the profile of the

water surface at the outer bank may actually rise near the middle point of the bend, causing the water near this bank to flow upstream for a short distance in this vicinity. The eddy thus formed is called a "pressure eddy."

At the lower end of the curve, the water surface along the inner bank, which has dropped below the average gradient, must return to that gradient, either by rising or by flattening out until the average gradient falls to it. Ordinarily the water line at this point is almost level, resulting in a region of low velocity.

The water at the outer bank, no longer piled up by centrifugal force, flows back to the central and inner parts of the stream, causing the outer water line to fall to the elevation of the average stream gradient. This increase in slope causes an acceleration of the water near the outer bank at the end of the curve. Since the water which is thus accelerated already has the highest velocity of any in the cross section, having been thrown from the convex bank toward the concave by centrifugal action, its velocity becomes conspicuously high as the curve adjusts itself to the tangent.

VELOCITIES MEASURED

In this investigation velocities were measured with a Price current meter. The water moved fastest near the inner bank above the bend, and near the outer bank below the bend. The highest velocities of all were found near the outer bank at Section 4. Low velocities were noted on the opposite sides of the river from the regions of high velocity. A large pressure eddy, indicated by the velocity vectors in Fig. 2, was evident near the concave bank at the middle point of the bend. This distribution of velocities is entirely in accord with the theoretical distribution that has been outlined.

Allowing for observational errors, the longitudinal profiles of the water surface, all of which are similar to that shown in Fig. 3, are shaped exactly as would be expected from the velocity distribution and the curvature of the stream.

MOVEMENT OF BEND

Comparison of the map of the bend in Fig. 1, made by us in 1933, with that made by D. L. Yarnell, M. Am. Soc. C.E., in 1925 shows that during these eight years the bend has retained its shape but has moved downstream about 100 ft. This movement has been due to the erosion of the left bank of the stream at and above the middle point of the bend, and to the deposition of material on the right, or north bank, as shown on the map. This deposition of sediment is in the region of the pressure eddy along the outer bank, where the velocity of the current is very greatly reduced. The tendency of bends to move downstream, noted by many observers, seems verified by this investigation. Such a general rule, however, should be applied to a given bend only after a study of actual conditions at the site.

THE BEND IN THE IOWA RIVER, NEAR IOWA CITY, ON WHICH THE INVESTIGATION WAS MADE

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

An Analysis of Deformation in Tension

By ALBIN H. BEYER, M. AM. SOC. C.E.
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AN analysis of the deformations which occur in any prismatic body subjected to an axial tensile load is not only interesting but essential to a clear understanding of the tensile phenomenon and the laws of elastic failure of engineering materials. The following simplified and illustrative analysis can be readily followed by anyone who understands Hooke's Law and internal stresses.

As ordinarily presented, the tensile stress distribution in a square prism is as given in Fig. 1(a), where P is the applied tensile load; A is the area of the cross section of the prism; and S equals $\frac{P}{A}$, the tensile stress parallel to the applied load. In Fig. 1(b) is shown the stress distribution on the four planes all inclined at an angle, θ, to the transverse cross section of the prism. The normal and tangential stress components which act on

these four planes are S_n and S_s, and their values are found by the equations:

$$S_n = S\cos^2\theta \dots\dots\dots\dots [1]$$

$$S_s = S\sin\theta\cos\theta \dots\dots\dots\dots [2]$$

When θ equals 45 deg, the shear stress component, S_s, reaches its maximum value and amounts to $\frac{S}{2}$. The normal stress component, S_n, acting on this plane is also equal to $\frac{S}{2}$.

By analyzing the deformation of the pyramid *Oabcd* in Fig. 1(*b*), the deformations which take place can be determined; first, that due to the normal stresses acting alone on the four inclined planes, and second, that due to the shearing stresses acting alone on the same planes, all as shown in Fig. 2(*a*) and (*b*).

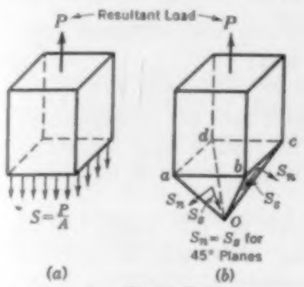

FIG. 1. UNIT STRESS DISTRIBUTION

(*a*) On Planes Parallel to the Load. (*b*) On Planes Inclined to the Load

In Fig. 2(*a*) the stress distribution is the exact equivalent of the principal stress distribution in Fig. 3(*a*), and in Fig. 2(*b*) the stress distribution is the exact equivalent of the principal stress distribution in Fig. 3(*b*). The combined effect of the two stress distributions of Fig. 3(*a*) and (*b*) is that of simple tension; that is, Fig. 3(*a*) and (*b*) if combined would be equivalent to Fig. 1(*a*).

Strains produced by the normal stress components, S_n equal to $\frac{1}{2}S$, are the same in all directions. From Fig 3(*a*) they are equal to

$$+\ \frac{S}{2E}\left(1 - \frac{2}{m}\right)\dots\dots\dots\dots [3]$$

where $\frac{1}{m}$ is equal to Poisson's ratio, and E is the modulus of elasticity in tension or compression. The corresponding deformation is shown in Fig. 4.

Strains which are produced by the tangential or shear stress components, S_s, equal to $\frac{S}{2}$, acting alone on the inclined planes, vary with the direction of the load. In the direction of the applied load, P, the strain may be computed from the principal stresses as shown in Fig. 3(*b*) and is equal to

$$+\ \frac{1}{2}\cdot\frac{S}{E}\left(1 + \frac{2}{m}\right)\dots\dots\dots\dots [4]$$

In a lateral direction, that is, at right angles to the load, the strain is equal to

$$-\ \frac{S}{2E} - \frac{S}{2mE} + \frac{S}{2mE} = -\ \frac{1}{2}\ \frac{S}{E}\dots\dots [5]$$

In the direction of the applied tensile load the resultant strain is the sum of the strains resulting from the stress distributions, Fig. 3(*a*) and (*b*), and is found by adding Equations 3 and 4, which equal $+\ \frac{S}{E}$. The corresponding strain in a lateral direction, found by adding Equations 3 and 5, amounts to $-\ \frac{S}{mE}$.

For stress distributions within the elastic limit of the material and for an assumed value of $^1/_3$ for Poisson's ratio, the shear stress components on planes inclined at 45 deg to the load account for five-sixths of the axial tensile strain and for all the net lateral tensile strain, from Equations 3 and 4, and from Equations 3 and 5, respectively.

This analysis clearly indicates that both the elastic limit and the yield point in tension must be controlled by the elastic limit and yield point of the material in shear. When the material is stressed to its elastic limit and plastic deformations are developed, Poisson's ratio increases and tends to approach the value of $^1/_2$, and consequently when plastic flow sets in, all the tensile deformations are due to the shear stress components on the inclined planes. In addition, this analysis shows that the theory of elastic failure by maximum shear stress also applies to the case of pure tension, all of which is in accordance with modern concepts of the elastic failure of engineering materials.

Failure to distinguish between elastic and ultimate failure has always created considerable confusion among engineers. To the designer, elastic failure is more important than ultimate failure. This is especially true for those machine parts which in service are subjected to repeated shocks and fatigue, for under such conditions elastic failure in a specimen subjected to a tensile load frequently progresses rapidly into ultimate failure. The phenomenon of ultimate failure is complicated; and the stress distribution in the zone of failure also is complex on account of the local contraction in the area of the cross section, which usually develops in that zone. Under such conditions it is almost impossible to determine accurately the stress distribution at ultimate failure. The values

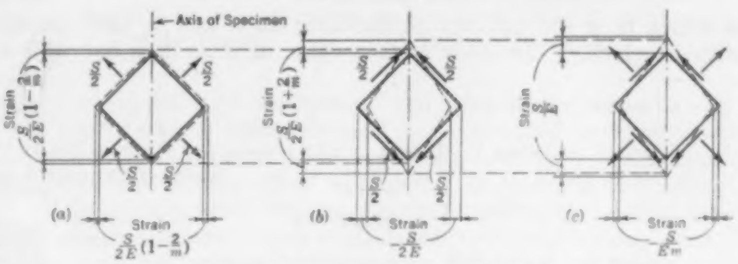

FIG. 4. STRAINS DEVELOPED IN ANY VERTICAL PLANE

(*a*) Strains Due to Normal Stresses, $\frac{S}{2}$, Acting Alone

(*b*) Strains Due to Shearing Stresses, $\frac{S}{2}$, Acting Alone

(*c*) Combined Effect of Normal and Shearing Stresses

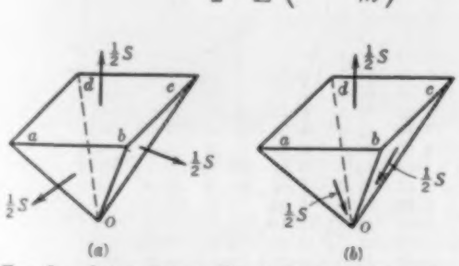

FIG. 2. ANALYSIS OF DEFORMATION OF THE PYRAMID IN FIG. 1(*b*)

(*a*) Normal Stress Components with Pure Tension in All Directions. (*b*) Tangential Stress Components, with Pure Shear in All Planes Parallel to the Load

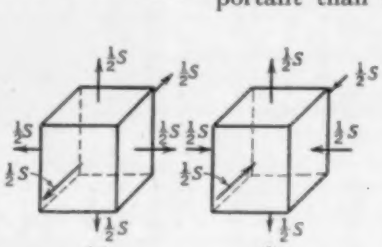

FIG. 3. PRINCIPAL STRESSES

(*a*) Corresponding to Normal Stress Components in Fig. 2 (*a*). (*b*) Corresponding to Tangential Stress Components in Fig. 2(*b*)

for ultimate strength that are given in handbooks at best must be considered nominal stresses, since they are not the actual stresses at which the material ruptures.

Length of Hydraulic Jump Investigated at Berlin

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IN 1927 a series of experiments was conducted in Berlin that considerably extended the range of data concerning the hydraulic jump. The results were published as Bulletin No. 6 of the Hydraulics Laboratory of the Technical University at Berlin by Dr.-Ing. Kurt Safranez. An article also appeared in *Bauingenieur* for 1929, Nos. 37 and 38. A complete English translation of this article is on file in the Engineering Societies Library in New York, N.Y. It is thought that a report of these tests has never been published in English, and therefore that a brief summary of their results may be helpful.

Previous experimenters had explored the relationship existing between the depth of tailwater after the jump and the other properties, discharge, velocity, and depth of the shooting flow before the jump, for values of the flow index, R, as high as 9, expressed by the equation:

$$R = \frac{v_1}{v_c} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad [1]$$

in which v_1 is the velocity of shooting flow, and v_c the critical velocity. In these tests a value of R as high as 19.10 was reached, with the result that the validity of the formula,

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}} \quad . \; . \; . \; . \; . \quad [2]$$

was further very satisfactorily confirmed. In this formula, d_2 is the tailwater depth; d_1, the depth of shooting flow; and g, the acceleration of gravity.

It was also found that a slight modification of the approximate Merriman formulas resulted in equations having an error of not more than 18 per cent, and for all values of R above 5, one of less than 5 per cent. The proposed approximate equations are:

$$d_2 = 0.240 v_1 \sqrt{d_1} \text{ (in feet)} \quad . \; . \; . \; . \; . \; . \; . \quad [3]$$

$$d_2 = 0.240 \frac{q}{\sqrt{d_1}} \text{ (in feet)} \quad . \; . \; . \; . \; . \; . \; . \; . \quad [4]$$

in which q is the discharge in cubic feet per foot of width. A comparison of the experimental results with the values of d_2 calculated by various formulas is given in the accompanying Table I.

From the careful measurements of the length of the hydraulic jump made by Dr.-Ing. Safranez, its terminus being arbitrarily defined as the point at which no return current could be detected with a coloring pipette, Prof.-Dr. A. Ludin concludes that "under all conditions of flow, the shooting stream dilates, if laterally confined, at practically the same angle, α, whose tangent is approximately $\frac{1}{4.5}$." More exactly, he finds that

$$\frac{d_2}{l} = \frac{1}{4.5} - \frac{1}{6R} \text{ (approximately)} \quad . \; . \; . \quad [5]$$

in which l is the length of the jump. The accuracy of this equation increases with the value of R.

TABLE I. DEPTH OF THE HYDRAULIC JUMP AS FOUND BY EXPERIMENT AND BY FOUR FORMULAS

Width of Channel, b = 49.9 cm

					(1) Momentum Formula, $d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}}$			(2) Constant Energy Formula $d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g}$			(3) Merriman's Formula $d_2 = \sqrt{\frac{2}{g}}\sqrt{v_1^2 d_1}$						(4) Revised Merriman Formula $d_2 = \sqrt{\frac{1.85}{g}}\sqrt{v_1^2 d_1}$		
						Error (from the observed)			Error			Error						Departure from Formula 1	
Test No.	Q_1 Liters per Sec	d_1 Cm	v_1 Cm per Sec	Observed d_2 Cm	Calculated d_2 Cm	Cm	%	Calculated d_2 Cm	Cm	%	Calculated d_2 Cm	Cm	%	$v_c = \sqrt{gd_1}$ Cm per Sec	$R_1 = \frac{v_1}{v_c}$	$W = \frac{d_2}{d_1}$	Calculated d_2 Cm	Cm	%
1	36.44	5.70	128.1	11.2	11.3	−0.1	−0.89	12.3	−1.1	−8.95	13.8	−2.6	−18.80	74.7	1.72	1.97	13.3	−2.0	−17.65
2	34.20	5.00	137.1	11.4	11.6	−0.2	−1.72	13.2	−1.8	−13.62	13.8	−2.4	−17.40	70.0	1.96	2.28	13.3	−1.7	−14.70
3	36.95	4.41	167.8	13.8	13.9	−0.1	−0.72	17.9	−4.1	−22.90	15.9	−2.1	−13.20	65.7	2.55	3.13	15.3	−1.4	−10.07
4	10.43	1.90	110.0	5.9	5.9	±0.0	±0.0	7.7	−1.8	−23.40	6.8	−0.9	−13.23	43.2	2.56	3.10	6.6	−0.7	−11.85
5	31.80	3.87	164.6	12.7	12.8	−0.1	−0.78	16.9	−4.2	−24.85	14.6	−1.9	−13.00	61.6	2.67	3.28	14.1	−1.3	−10.10
6	26.23	3.20	164.2	11.6	11.8	−0.2	−1.69	16.4	−4.8	−29.27	13.3	−1.7	−12.78	56.0	2.93	3.62	12.8	−1.0	−8.42
7	36.95	3.23	229.1	17.0	17.0	±0.0	±0.00	29.6	−12.6	−42.55	18.6	−1.6	−8.61	56.3	4.08	5.27	17.9	−0.9	−5.3
8	15.97	1.60	201.0	10.4	10.7	−0.3	−2.80	22.1	−11.7	−53.00	11.5	−1.1	−9.57	39.6	5.03	6.50	11.0	−0.3	−2.81
9	27.42	2.28	241.1	15.3	15.3	±0.0	±0.00	31.8	−16.5	−51.80	16.4	−1.1	−6.72	47.3	5.10	6.72	15.8	−0.5	−3.26
10	20.90	1.90	220.4	13.1	12.8	+0.3	+2.34	26.6	−13.5	−50.75	13.7	−0.6	−4.38	43.2	5.12	6.90	13.2	−0.4	−3.12
11	38.10	2.25	339.0	21.7	21.9	−0.2	−0.91	60.8	−39.1	−64.30	22.9	−1.2	−5.24	46.9	7.25	9.65	22.1	−0.2	−0.91
12	26.18	1.60	328.0	17.5	17.9	−0.4	−2.24	56.5	−39.0	−69.00	18.7	−1.2	−6.42	39.6	8.28	10.94	18.0	−0.1	−0.56
13	32.80	1.85	355.5	20.1	21.0	−0.9	−4.28	66.2	−46.1	−69.60	21.8	−1.7	−7.80	42.6	8.36	10.86	21.0	−0.0	−0.00
14	8.27	0.73	225.0	8.3	8.4	−0.1	−1.19	26.5	−18.2	−68.75	8.7	−0.4	−4.60	26.8	8.40	11.36	8.3	−0.1	−1.19
15	36.00	1.85	390.0	22.8	23.0	−0.2	−0.87	78.3	−55.5	−70.85	23.9	−1.1	−4.61	42.6	9.17	12.32	23.1	−0.1	−0.43
16	14.68	1.00	294.2	12.7	12.8	−0.1	−0.78	45.2	−32.5	−72.00	13.3	−0.6	−4.51	31.3	9.46	12.70	12.8	−0.0	−0.00
17	22.94	1.35	342.0	17.0	17.3	−0.3	−1.73	65.2	−48.2	−73.90	18.2	−1.2	−6.60	35.7	9.60	12.60	17.3	−0.0	−0.00
18	34.98	1.65	425.0	24.2	23.8	+0.4	+1.67	93.7	−69.5	−74.20	24.6	−0.4	−1.63	40.1	10.64	14.66	23.7	−0.1	−0.42
19	25.07	1.30	386.0	18.8	19.2	−0.4	−2.08	77.3	−58.5	−75.70	19.8	−1.0	−5.06	35.7	10.90	14.46	19.1	−0.1	−0.52
19a	33.45	1.30	516.0	25.7	25.9	−0.2	−0.77	136.9	−111.2	−81.30	26.6	−0.9	−3.38	35.7	14.50	19.76	25.6	−0.3	−1.16
20	21.35	0.95	450.0	18.5	19.3	−0.8	−4.15	104.0	−85.5	−82.30	19.8	−1.3	−6.57	30.5	14.70	19.47	19.1	−0.2	−1.03
21	32.20	1.25	517.0	24.5	25.4	−0.9	−3.54	137.1	−112.6	−82.30	26.1	−1.6	−6.13	35.0	14.80	19.60	25.2	−0.2	−0.79
22	21.55	0.95	455.0	19.0	19.5	−0.5	−2.57	106.3	−87.3	−82.10	20.0	−1.0	−5.00	30.5	14.90	20.00	19.2	−0.3	−1.53
23	15.10	0.70	432.3	15.6	16.0	−0.4	−2.50	96.0	−80.4	−83.75	16.3	−0.7	−4.30	26.2	16.5	22.28	15.7	−0.3	−1.87
24	16.19	0.73	444.0	16.6	16.8	−0.2	−1.19	101.2	−84.6	−83.60	17.1	−0.5	−2.92	26.8	16.60	22.73	16.5	−0.3	−1.78
25	24.15	0.95	509.0	21.7	21.9	−0.2	−0.91	133.1	−111.4	−83.80	22.4	−0.7	−3.12	30.5	16.70	22.85	21.5	−0.4	−1.82
26	17.80	0.71	502.5	18.2	18.8	−0.6	−3.19	129.2	−111.0	−85.90	19.1	−0.9	−4.72	26.4	19.10	25.62	18.4	−0.4	−2.12

For overflow weirs with efficiencies, m, of from 0.80 to 0.85, Professor Ludin deduces that

$$l = \sqrt{8mg}\ \sqrt{h_1d_1}\ \text{(approximately)}$$
$$= 8\ \sqrt{h_1d_1}\ \text{(approximately)} \quad \ldots\ldots \quad [6]$$

and, if C equals 0.75 in the formula, q equals $\frac{2}{3}\ CH\sqrt{2gH}$, and

$$l = 6\sqrt[4]{H_1{}^3h_1}\ \text{(approximately)} \quad \ldots \quad [7]$$

in which h_1 is the velocity head of the shooting flow, and H_1 is the head on the weir crest. The equation for sluices would become

$$l = 8\sqrt{h_1cD_1}\ \text{(approximately)} \quad \ldots \quad [8]$$

c being a contraction coefficient, and D_1 the sluice opening.

These formulas, derived at the river hydraulics laboratory of the Technical University of Berlin under the immediate supervision of Professor Ludin, whose reputation in the field of hydro-electric development is world-wide, may prove both acceptable and useful to those who may at some time be confronted with the problem of designing a tumble basin or a flume involving the hydraulic jump.

What Is Plate Action?

By L. Sandberg

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Even in a field as widely studied as that of concrete construction there may be problems commonly regarded as mysteries, as, for instance, the question of what takes place in a two-way reinforced slab supported on all four sides. When confronted with the design of such a panel, the engineer's usual procedure is to split the total load in two parts, proportionate to a certain power of the ratio between the panel sides. This done, the partial loads are applied to the slab for each direction, as for a one-way construction. Furthermore, with

Plate Action in a Loaded Test Panel 4 1/2 In. Thick and 16 1/2 Ft Square

The Corner Has an Upward Deflection of 7/8 In. with a Superimposed Load of 665 Lb per Sq Ft. To Keep the Corners Anchored Down Under This Load Would Require at Each a Force of Nearly 18,000 Lb

this load and with the actual span of the panel in the direction considered, a moment factor is used that is lower than the one-way factor. The difference, it is said, is due to "plate action," or to "redistribution of stress." These terms, however, are rather vague; they do not tell exactly how the plate acts or how the stress is redistributed.

It may be clearer to refer to the "torsional moments" that are produced because the slab bends in two directions. Designs are based on the assumption that the panel is made up of a system of beams running at right angles to each other. Before any load is applied, the cross section of every one of these beams must necessarily be rectangular. But, as is known, the slab when loaded will take a shape somewhat similar to that of a saucer. Then only one beam in each direction will be able to remain rectangular, and that is the one running along the center line. All the other beams will acquire sections like parallelograms, with their angles varying over the entire length; otherwise the surface of the slab

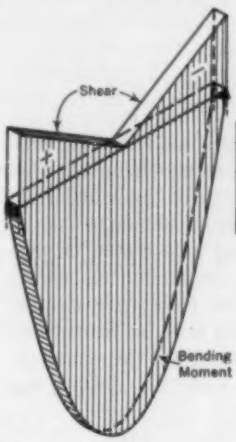


Fig. 1. Uniformly Loaded Beam

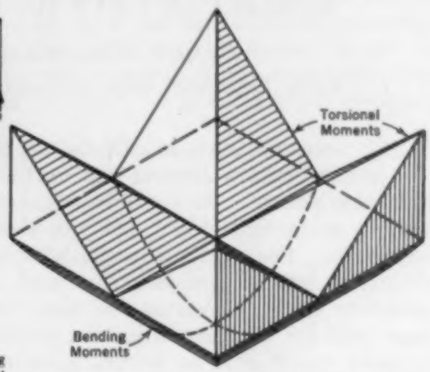


Fig. 2. Uniformly Loaded Panel

Approximate Bending and Torsional Moment Diagram; a Section Parallel to the Side of the Panel Should Be Part of a Parabola

could not stay continuous. In other words, when beams are running crosswise and are connected so that they form a two-way panel, not only will they bend when loaded, but also they will twist. In addition to bending moments there will be torsional moments. These obtain their maximum value where two neighboring beams have the greatest difference in deflection, or where the surface has its greatest slope, that is, near the corners.

When a panel is supported on all four sides and reinforced in two directions, its carrying capacity is made dependent on an additional dimension. Whereas a beam (or a one-way slab) is governed by its cross section and one span length, a two-way panel is determined by the magnitudes of its cross section and two span lengths, and the moment diagram for a two-way panel not only will be a three-dimensional diagram but also will show an additional kind of strain.

In a uniformly loaded beam (Fig. 1), the end reaction together with the load produces shear. This load again causes normal stresses, positive and negative, in the cross section and produces a bending moment. The curve of the bending-moment diagram is determined from the increment of shear.

Similarly, in a uniformly loaded two-way panel, shear produces bending moments, and these in turn produce torsional moments, positive and negative. The shape of the torsional-moment diagram is determined from the increment of bending moments. One such diagram is shown on top of the panel in Fig. 2. It is only approximately correct, however, as a section parallel to the side of the panel should be part of a parabola. In both figures moments are drawn to scale for span lengths of 16 ft.

The substitution of an ordinary pyramid, as shown, was suggested by Dr. H. Marcus in his book, *Vereinfachte Berechnung biegsamer Platten* ("Simplified Computation of Flexible Slabs"), Berlin, 1929. The height was taken so that it gave the pyramid a volume equal to that of the correct diagram.

The test panel in the photograph was built by the Republic Fireproofing Company, Inc., at Bound Brook, N.J. It shows clearly how torsional moments cause the corners to bend upward. The book by Dr. Marcus contains approximate formulas for determining bending and torsional moments, making it fairly easy to undertake a more accurate computation if that should be required. Substitution moments, composed of bending and torsional moments, may be introduced or separate steel may be put in for each kind of moment. For large, oblong, freely supported panels, an investigation might be profitable. In the majority of cases, however, the methods in use in this country will give sufficient quantities of reinforcement. The shape of the complete spatial-moment diagram might give a hint as to the proper arrangement of this reinforcement.

Inflection Points in Frames Under Vertical Loads

By OTTO GOTTSCHALK
CIVIL ENGINEER, BUENOS AIRES, ARGENTINA

THE accurate statical calculation of skeleton frameworks centers around the correct position of the inflection points. As small inaccuracies in their location lead to magnified errors, rules of thumb are utterly inadequate. For vertical loads, leaving side sway of the structure out of consideration and assuming as usual members each of constant section, the following simple expedient for locating inflection points will be found extremely useful and as accurate as most of the complicated, supposedly exact methods. For the underlying

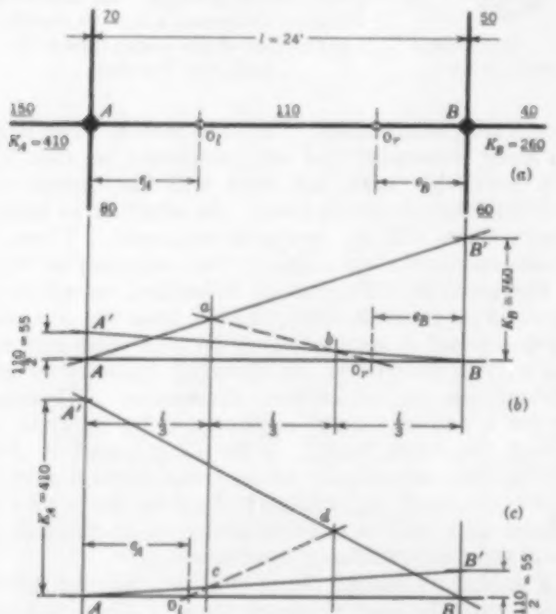


FIG. 1. GRAPHIC DETERMINATION OF INFLECTION POINTS

theory and the example of a symmetrical frame reference may be made to *Der Bauingenieur*, Berlin, December 16, 1932, page 620.

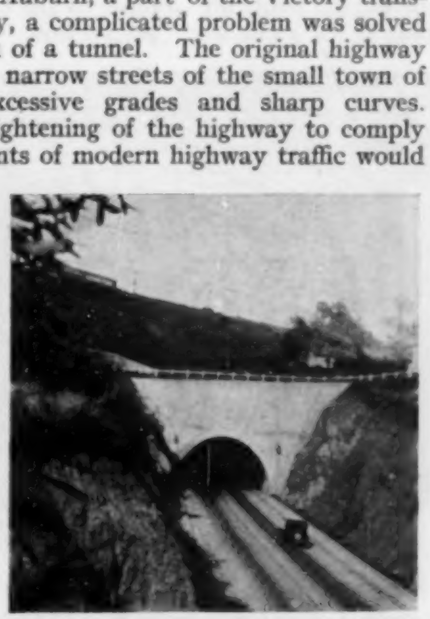
Let AB in Fig. 1(a) be any member of a skeleton, the inflection points of which are required. The underscored members (such as 110 for beam AB) indicate the assumed stiffness factor, $\overline{K}$ (which is the moment of inertia divided by the length). Summing about each joint, $K_A = \underline{410}$, the total stiffness of all the members joining at A; and $K_B = \underline{260}$, for those at B.

Using any convenient vertical scale, in Fig. 1(b), $BB' = K_B = 260$, and $AA' = \frac{110}{2} = 55$. Draw AB' and BA' and locate their third points, a and b. Then the straight line through a and b crosses AB at the right inflection point, O_r. Similarly, in Fig. 1(c), making $AA' = k_A = 410$, and $BB' = \frac{110}{2} = 55$, the left inflection point, O_l of AB is found at the intersection of the straight line through the third points, c and d of AB' and BA' with AB.

Tunnel Solves Highway Relocation Problem

By A. A. EREMIN, ASSOC. M. AM. SOC. C.E.
ASSOCIATE BRIDGE DESIGNING ENGINEER, CALIFORNIA DIVISION OF HIGHWAYS, SACRAMENTO, CALIF.

IN improving the California State Highway between Sacramento and Auburn, a part of the Victory transcontinental highway, a complicated problem was solved by the construction of a tunnel. The original highway passed through the narrow streets of the small town of Newcastle with excessive grades and sharp curves. Widening and straightening of the highway to comply with the requirements of modern highway traffic would have involved costly town alterations; therefore the new highway was relocated and a tunnel constructed through the ridge under the main line of the Southern Pacific Railway and a part of the adjacent town.



SOUTH PORTAL OF NEWCASTLE TUNNEL, CALIFORNIA

This more direct route through the hill shortened the highway by saving 1,418 ft in length and eliminating curves whose central angles totaled 599 deg (Fig. 1). Moreover, the local town traffic, which is especially heavy during the fruit-shipping season, has been considerably

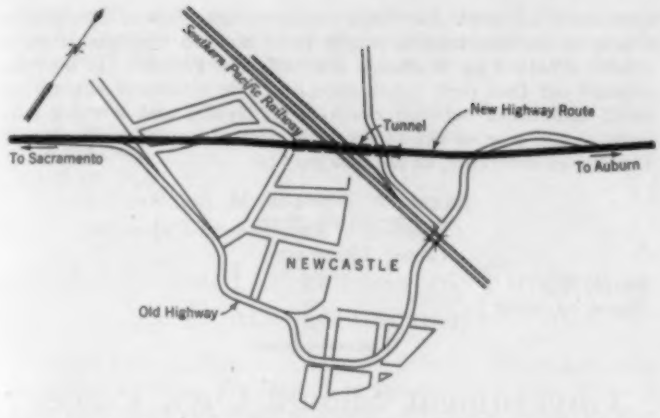


Fig. 1. Highway Relocation Problem at Newcastle, Calif.

relieved by diversion of the through traffic from the business district.

Rising toward Auburn on a grade of 1.92 per cent, the tunnel is 531 ft long. Its section is shown in Fig. 2. The ground which it penetrates consists of granite of variable hardness cut by numerous shear planes and minor fault lines and partially decomposed and disintegrated at the shear planes and the tunnel portals. The maximum depth of material above the roadway is 84 ft.

A three-center intrados was used in preference to the semicircular shape because it dissipates the reverberation or echo of traffic noises. Also, since the section has a greater height, it has a smaller frictional resistance to the natural draft carrying the vitiated air out of the tunnel. Moreover, the three-center arch lining has structural advantages in supporting the sheared rock.

Excavation was begun by blasting a crown heading 7 by 8 ft in section from each end of the tunnel. The crown heading at the south end was followed by two side bottom drifts, also 7 by 8 ft in section. At intervals of about 75 ft the crown heading and the side drifts were connected by inclined shafts about 4 ft square. The crown drift was widened to full arch span by blasting and stoping the material into mine cars on tracks in the side drifts. The excavated material was hauled to the spoil pile at the south approach.

Concrete in the side-wall lining was poured by gravity to a height of 15 ft from $^3/_4$-yd mine cars operating on tracks laid on top of the core. The crown sections, which were 20 to 40 ft in length, were placed with a concrete gun. The overbreak in the excavation was filled with the same concrete as that used in the lining. After the concrete lining was in place, the core was broken by powder blasting, and the material was removed with a power shovel and transported by trucks.

Unit prices included $120 per lin ft for the tunnel excavation and $110 per lin ft for the reinforced concrete lining in place. Actual work on the tunnel began on November 28, 1930, and removal of the core was completed on November 17, 1931. The tunnel was designed and built by the California Division of Highways under the general direction of C. H. Purcell, Assoc. M. Am. Soc. C.E., State Highway Engineer.

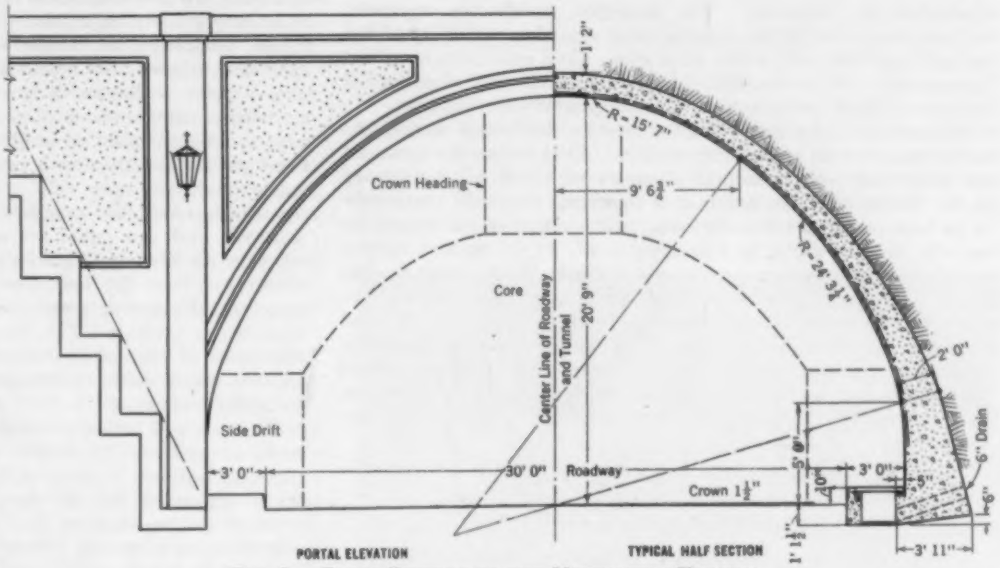


Fig. 2. Cross Section of the Newcastle Tunnel
The Three-Center Shape Reduces Reverberation of Traffic Noises

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

The Geologist Can Aid the Engineer

To the Editor: The article by Mr. Crosby on the "Geology of the Fifteen Mile Falls Development," in the January issue, presents clearly the manner in which a geological approach may contribute toward the solution of problems involved in the location, design, and construction of dams. The civil engineer is particularly interested in the properties of matter such as rock, clay, or sand, when they are within or adjacent to the construction lines of the structure. However, he is interested in obtaining an understanding of the larger geologic picture when such an understanding will help him in visualizing his own particular problem, and will aid him in its solution. It is necessary, therefore, for the engineer to obtain the right kind of geological data in as much detail as possible.

There is a wide range of behavior in the different rock types forming a part of the earth's crust. Some rocks are soft; others are hard, and no one term is descriptive of any particular type of the several classes of rock. Many of the younger or more recent sedimentary rocks in the West are very friable, highly porous, and have never been hard in the sense that the term is understood in the East. Such formations present difficult problems for the

engineer and constructor, and the geologist may contribute much to the solution of them.

It is not generally known that crystalline rocks do not always retain the same sound character within their formational limits. Hard granites can change into a soft and semi-plastic mass, which will be stable as long as it is confined. When excavating is done, however, this material may swell, flow into the excavated space, and create a serious condition. Likewise hard rocks, such as schists or granites, sometimes scale excessively as a result of air slacking or the release of confined strains.

The geology of a localized area sometimes will not show natural conditions unfavorable to the construction of an engineering structure, such as a reservoir. However, a study of the geology over the larger catchment area might disclose geological conditions that would seriously affect successful operation of the reservoir. Geologic studies of the larger area play a particularly important part in the location of tunnels, because such projects generally extend for miles. An understanding of both the surficial and underground geology of a region is therefore important in the selection of the most economical location, which often is not along the shortest path between shafts or portals either in plan or profile. Deviations from ideal locations are often necessary in order to avoid a deeply buried valley, or a rock formation or structural condition that might cause serious difficulties.

Conclusions derived from geologic studies are necessarily largely qualitative in character. The geologist, unlike the engineer, cannot always rely on the continuity or sameness in quality of the natural materials with which he works. Civil engineers, however, have contributed substantially to the quantitative side of geological problems in their investigations of underground water supplies.

Geological studies make their greatest contribution during the earlier stages of an engineering project. At this time the geologist can point out certain natural obstacles which, if not considered in the design or construction of a structure, may add materially to its cost or cause serious trouble. As a result of his studies he may be able to assist in relocating a site or to suggest helpful modifications in design or method of construction. And finally, he may be able to present his judgment as to the feasibility of the project in question. At this stage the geologist must bear in mind the fact that there are very few adverse conditions that cannot be overcome by modern engineering and construction methods, where cost is not necessarily a controlling factor. It is here, however, that the engineer assumes his right of final judgment and responsibility.

FRANK E. FAHLQUIST, Assoc. M. Am. Soc. C.E.
Assistant Engineer, Metropolitan District Water Supply Commission

West Brookfield, Mass.
March 21, 1934

Differential Two-Liquid Gages and Specific Gravities

TO THE EDITOR: In the January number of CIVIL ENGINEERING, Messrs. Eagle and Wilson cast considerable suspicion on the widespread practice of using specific gravities in arriving at the desired "differential head" in Venturi meters and Pitot tubes, when measuring losses of head, and in similar cases. It is therefore interesting to note in Vol. 47 of the TRANSACTIONS of the Society (1902), pages 81 and 336, a comment by Williams, Hubbell, and Fenkell as follows: "For kerosene (above water columns) the multiplication is about 4 per cent greater than is accounted for by the difference in specific gravities, and in the case of sperm oil, . . . the increase in the factor of multiplication was as much as 10 to 12 per cent.

"This increase of multiplication seems to be due to the adhesion of the oil to the glass, and very probably may vary with the size of the tube; but, as in the writers' experiments, the oils were calibrated in the respective gages by comparison with a water column, it was not considered necessary to investigate the matter further."

It seems probable that further investigation with the water column would have shown decreases as well as increases in the factor of multiplication to the same percentage extent. Moreover, one is led to wonder what effect non-recognition of the variable effects of surface tension might have had on the quantitative results obtained by Williams, Hubbell, and Fenkell. It may be pointed out that their conclusions on curve resistance depend on small differences between much larger heads, and a small percentage one way or the other might almost annihilate or double the desired difference, as the case may be.

ERNEST W. SCHODER, M. Am. Soc. C.E.
Professor of Experimental Hydraulics
Cornell University

Ithaca, N.Y.
March 14, 1934

Government Should Own Power Utilities

TO THE EDITOR: In his article, "An Equitable Theory of Governmental Ownership and Operation," in the March issue, Mr. McDonald displays the attributes of a military strategist in that he readily changes his position. Starting with a general discussion which concedes no place for governmental authority in utility operation, his first conclusion is apparently for government subsidy of strictly private enterprise. Perhaps this was written prior to recent disclosures of congressional committees. Dropping this idea undeveloped, Mr. McDonald outlines a mutual ownership plan, which provides for the gradual amortization of private investment from rates under a plan of private management and public supervision. A question unanswered is whether private management should prevail after a condition of complete amortization is reached.

I readily admit that in public-owned utilities the contributions of taxpayer and rate-payer are not always equitably balanced but, contrary to Mr. McDonald's assumption, the taxpayer is more often than not the one who benefits. Free water service to municipal departments and free fire protection are often burdens upon water utilities. The long list of "tax free" towns cited by advocates of municipal ownership is a convincing indication of success, but it does not represent an equitable distribution of the costs of government.

Opponents of public ownership continually stress the fact that governmental utilities do not pay taxes. Many if not most of them actually do make contributions to current funds in lieu of taxes. However, of far more importance is the fact that the municipal rate-payer not only pays fixed and operating costs but also buys an equity in the utility. Bonds are almost invariably amortized faster than the plant depreciates. Under public ownership the interest charge is gradually reduced, while under private ownership the rate base grows and grows. In a recent study of important Pacific Coast utilities I found that the sum of payments in lieu of taxes plus net income (public profit) of municipal utilities was from two to five times as much as the taxes paid by private companies.

A large majority of American water-works utilities are under public ownership, and their management is generally successful. In fact, the rule is that the decrepit, inadequate water systems of the country are those in private hands. What is to keep public management of electric power from being just as efficient as that of water works?

Engineers in some parts of the country will assert that municipal power plants are complete failures, but those who live on the Pacific Coast and are really familiar with the situation in Los Angeles, Tacoma, and Seattle have evidence to the contrary. The plants in these cities charge low rates, make adequate allowances for depreciation, and still earn a substantial net income, which goes into plant extensions and bond retirement.

Few believers in public ownership are so extreme in their views as to advocate confiscation of, or even destructive competition with private plants, in spite of the fact that the large power companies have recognized no such ethics in their dealings with the public or with smaller competitors. As a substitute for Mr. McDonald's plan, the public purchase of private plants, one by one as public sentiment and expediency dictate, would seem a more feasible course. It is exactly what has taken place in the field of water works. Under public management, mistakes and inefficiency will

occasionally occur, but it is here predicted that by 1954 most of Mr. McDonald's fears will be found unjustified, and power will be universally a public-owned utility.

John W. Cunningham, M. Am. Soc. C.E.
Consulting Engineer

Portland, Ore.
March 26, 1934

Additional Methods of Finding Beam Deflections

To the Editor: The novel method of finding beam deflections presented by Ralph W. Stewart in the February issue of Civil Engineering is well worth attention. A partial listing of other methods may be of interest.

First of all, there is the classic method of determining the deflection of a loaded beam from the equation of the elastic line, often called the double integration method. We owe this method to Euler (1707–1783).

Today the method of moment areas, or area moments, has first place in most textbooks. The two forms of this method often presented are the conjugate beam method, sometimes called the method of elastic weights, and the slope deviation method. Some writers consider them the same method; others regard them as different methods. Hool and Kinne, in *Structural Methods and Connections*, credit the area-moment method to Professor Mohr and the conjugate beam method to Professor Greene. The conjugate beam method is described by H. M. Westergaard, M. Am. Soc. C.E., in a masterly paper in the *Journal of the Western Society of Engineers* for November 1921. The slope deviation method, often called the moment-area method, may be either mathematical or graphical.

The name of Castigliano is inseparably associated with methods of work. His treatise in French was published in 1879. A translation of this by Professor Andrews bears the title, *Elastic Stresses in Structures* (London, 1919).

The Maxwell-Mohr modification of the principle of virtual work equates the external work with the internal work or resilience. In *Advanced Mechanics*, published in 1932, Seely shows that the deflection of a point of any structure obtained by Castigliano's theorem is the same as that obtained by the Maxwell-Mohr principle of virtual work.

The advantages of the Frankel method (1875)—a dummy load of unity placed at the point where the deflection is desired—are well shown in *Deflections and Statically Indeterminate Stresses* by Clarence W. Hudson, M. Am. Soc. C.E. In *Engineering* for November 11 and 18, 1921, T. Thompson expresses the load intensity in a sine series and applies it to obtain the deflection of loaded beams.

John Case, in *The Strength of Materials* (London, 1925), determines the deflection of beams by harmonic analysis. He refers to papers by Professors Timoshenko and Inglis. The paper by Professor Inglis (*Proceedings of the Institution of Civil Engineers*, Vol. 217, 1924, pages 225–272) states that "nature intended deflection of girders with free ends to be studied by means of harmonic analysis." He also finds Macaulay's method extremely useful in dealing with beam problems where the bending moment is discontinuous.

In his *Strength of Materials*—Part 2, "Advanced Theory and Problems"—published in 1930, Timoshenko devotes a chapter to the subject, "Representation of the Deflection Curve by a Trigonometrical Series." Morley refers favorably to a paper by Dr. George Wilson "where the deflections are found in a novel graphical way." This paper may be found in the *Proceedings of the Royal Society of London*, Vol. 62 (1897–1898), pages 268–279.

The extent to which purely graphical methods can be applied is well shown in *Graphical Analysis* by William S. Wolfe, Assoc. M. Am. Soc. C.E. Influence lines for deflection have received considerable attention. Steinman, in the *Engineering Record* for November 25, 1916, says that "Every influence line is a deflection diagram."

An algebraic method proposed by Prof. J. B. Kommers (*Engineering News-Record*, January 2, 1919) is quoted by Maurer and Withey in their *Strength of Materials.*

Space will permit only an allusion to fixed points, characteristic points, and conjugate points. A veritable mine of information is the paper, "Moments in Restrained and Continuous Beams by the Method of Conjugate Points," by L. H. Nishkian and D. B. Steinman, Members Am. Soc. C.E., and the discussion thereon. (Transactions of the Society, Vol. 90, 1927, pages 1–206.) The discussion is by no means confined to conjugate points. Mention may also be made of the paper, "Analysis of Continuous Frames by Distributing Fixed-End Moments," by Hardy Cross, M. Am. Soc. C.E., published in the Transactions of the Society, Vol. 96, 1932, pages 1–156.

An exhaustive paper on the shearing deflection of beams by Prof. S. E. Slocum, entitled "A General Formula for the Shearing Deflection of Beams of Arbitrary Cross Section, Either Variable or Constant," may be found in the *Journal of the Franklin Institute*, Vol. 171, pages 365–389, April 1911.

Robins Fleming
American Bridge Company

New York, N.Y.
March 23, 1934

The Art of Bridge-Building in Tibet

To the Editor: In connection with Mr. Fugl-Meyer's article on bridge-building in China, which appeared in the February issue, the accompanying photograph may be of interest. It shows a well-constructed cantilever bridge at Yatung, in the Chumbi Valley, Tibet. The span appears to be about 50 ft long.

Bridge at Yatung, in the Chumbi Valley, Tibet

This type of bridge seems particularly suited to the conditions obtaining, since it requires neither iron for joints nor an elaborate plant for erection. Mule loads are the greatest live loads. I have been informed by the friend who supplied the photograph that some of the natives are adept in the rapid construction of small bridges.

Robert A. Sutherland, Assoc. M. Am. Soc. C.E.

Wellington, New Zealand
March 13, 1934

More Chinese Bridges

Dear Sir: The illustrations for Mr. Fugl-Meyer's interesting article on "Bridge-Building, an Art of Ancient China," in the February issue of Civil Engineering, show some of the old structures to be in need of repair. This, of course, is not very surprising, considering the length of time they have been in use. The traveler in China sees many examples of the skill of ancient

Chinese engineers, in the form of bridges, buildings, pagodas, walls, canals, and dikes, and in many cases these wonderful old structures have been allowed to fall into a state of partial or complete ruin. In most cases this is probably because no funds have been provided for their upkeep and repair.

The accompanying photograph of a stone bridge over the Ssü Ho, at Yen Chow-Fu, in Shantung, China, shows an old highway bridge that is in fair condition. It is uncertain how old the bridge is, but tablets erected near the end of it commemorate the repairing of the structure at different times. According to these tablets, the latest repairs were made more than one hundred and trict fifty years ago. From this it may be inferred that the several hundred years old.

Typical small Chinese bridges are illustrated in the other photographs accompanying this discussion.

C. O. CAREY, M. Am. Soc. C.E.
Associate Professor of Geodesy and Surveying, University of Michigan

Ann Arbor, Mich.
March 28, 1934

THREE CHINESE STONE BRIDGES
Above, Triple Arch Stone Bridge Over the Niu To Ho, at Wang Kue Tun; Upper Right, One of the Stone Lions Guarding the Entrance to Bridge at Lower Right; Lower Left, Stone Slab Bridge at Ku Ting Chun, Shantung (Note Surveyors at Work); and Lower Right, Old Multiple-Arch Bridge Over the Ssü Ho, at Yen Chow-Fu, Shantung

Education and the Contractor

DEAR SIR: I was very much interested in Colonel Crocker's article, "The Engineering Viewpoint," in the March issue. As far as my limited experience enables me to see, Colonel Crocker is exactly right in regard to the relation between contractor and engineer.

In the past the college training of many civil engineers has been decidedly incomplete without instruction in construction methods. I am glad to announce that at least one college faculty has realized this lack. For several years there has been at Iowa State College an elective course in construction methods, which covers fundamental construction operations and the application of construction machinery to engineering construction. In the past year, the entire civil engineering curriculum has been revised, and during the process it was wisely decided to make "C.E. 471" (Construction Methods) a required course. Now all the students can have the experience, before graduation, of learning the difference between a steam-hammer and a drop-hammer!

About eight months ago I had an interview with a U. S. District Engineer concerning an inspector's job on river work. When I mentioned that I had completed five years of college work, he disparaged the inspector's job by saying, "You ought to wait for an engineer's job. Inspection requires no engineering training; some construction experience and a little judgment are all that are necessary." I had always thought that construction of a project according to the plans was fully as important as the design itself, yet here was a man who would entrust the inspection of a structure to a person with no engineering training, and who differentiated between inspection and an "engineer's job."

This attitude is bound to change, however. If it does not change now, it will die with the older generation, and informed engineers will gradually replace "practical men" in the construction industry.

M. C. LORENZ, Jun. Am. Soc. C.E.
Construction Foreman, E.C.W. in Iowa, S.P. Camp No. 4

Johnston, Iowa
March 12, 1934

Junior Engineers Need Consideration

TO THE EDITOR: The article, "The Engineering Viewpoint," by Herbert S. Crocker in the March issue is astoundingly accurate in its analysis of the difficulties encountered in the construction industry. The causes of these difficulties, however, are analyzed from the point of view of an older man with conservative outlook and not from that of the younger man. Perhaps that is why he finds fault with the younger men instead of with the system under which young engineers work.

At the present time either the Government or large corporations are in charge of the construction industry. The young engineer is not an apprentice in a profession; on the contrary, he is a skilled workman in a trade. The reasons for this are many, ranging from the fact that age is a factor in obtaining a professional license, which is itself a trade outgrowth, to the overcrowding of the field by many men who are not engineers but merely persons who can memorize sufficient technical facts to get degrees from correspondence schools or colleges.

Professional engineers do not use young engineers as assistants but rather as employees to perform specific tasks. The opportunity of expression is not allowed the younger man; nor is the opportunity for obtaining advice often realized. In other words, professional courtesy is not practiced, and the young engineer's only recourse is to become autocratic in the small world he is allowed to govern. He cannot discuss general policies so he exercises his authority by requiring a contractor to put in the last drop of water necessary for a proper water-cement ratio or executing other needless detail.

That the average engineer in the field restricts the contractor cannot be denied. The practice is absurd and should be remedied. However, any contractor who finds this the case will do well to consider the circumstances under which the engineer works. If the engineer is inexperienced and not sufficiently able to judge the importance of the situation in question, then the contractor should appeal to a superior for a reversal of the decision, but this should be done in such a manner as to give the young engineer a practical lesson. However, it may be that the contractor is working with superintendents who themselves cannot realize the importance of specifications. If a contractor does not use proper care in the ordinary course of a day's work, he can hardly expect that a competent young engineer will allow slipshod work when a failure means a "call upon the carpet."

More mistakes are made because of the faulty relation between the design and execution of a construction project than because of the inability of young construction engineers. Although a good design is of prime importance, the young engineer cannot change any plans in the field without due approval from a superior. When that superior will not accept opinions from young men, the decision is made by a man who has the advantage of seniority but not necessarily a better understanding of the situation in question. Thus there is created a group of irresponsible young men who have ceased to aspire to a professional standing. This leaves in the making a trade that has been labeled "sub-professional" work.

The time required for obtaining a professional license forces the younger man to lose the hope of attaining the dignity of a professional status until he has become fairly mature. In the meantime he has married and assumed a standard of living lower than that of the usual professional worker because "sub-professional" work has not been deemed worthy of a substantial wage scale. The resulting psychological retrogression places restraints upon the young engineer's executive ability, and he allows his pent-up energy to be spent on detail. Is the young engineer therefore wholly to blame for his indiscreet methods, which have a deleterious effect on construction in general?

The young engineer is, then, an intelligent person who starts out with the proper motives and the possibility of being a useful worker and a credit to the profession, as those who have already advanced to that status have proved. He only asks the established engineers to remember their own start in the profession and to help the younger member instead of forgetting and philosophically saying, "Son, I'm older than you, I know better than you do. Do as I say!" Wouldn't it be just as easy to say, "What are your ideas, son?" Then analyze them. They may be worth consideration.

FRANCIS L. BROWN, Jun. Am. Soc. C.E.

Whitehall, N.Y.
March 15, 1934

Realignment Method of Flood Control on the Mississippi

TO THE EDITOR: The subject of flood control on the Mississippi is always of interest. Even if the present system proves as successful as it is hoped it will, there will still be a variable channel and periodic flooding.

A few years ago, a plan was proposed for a complete realignment of the river, dividing it into a few long reaches. There is distinct merit in the suggestion. The distance from Cairo to the Gulf can be shortened 40 per cent. The slope from Cairo to sea level below Natchez would be 0.54 ft per mile, and that from there to the sea, 0.25 ft per mile.

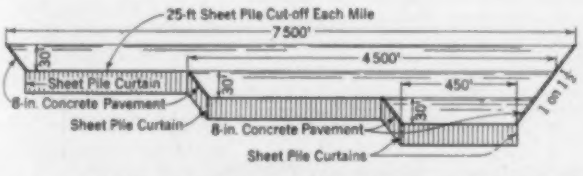


FIG. 1. PROPOSED SECTION FOR PERMANENT STRAIGHTENED CHANNEL FOR THE MISSISSIPPI FROM CAIRO TO THE GULF

A channel of the form shown in Fig. 1 would carry 80,000 cu ft per sec in the lower section, at a velocity of 3.75 miles per hour; 900,000 cu ft per sec in the lower two sections, with a velocity of 4.25 miles per hour; and 3,000,000 cu ft per sec in the full section, with a velocity of 4.45 miles per hour. Below Natchez the channel section would be trapezoidal. Extreme high water at Plaquemine would be only 16 ft above sea level, so that the connection to the present channel could be left permanently open without fear of damage.

The minimum recorded flow at Cairo of 70,000 cu ft per sec would assure a channel depth of 25 ft and an almost continuous depth of 30 ft or more. A high-water time of flow of $4^1/_2$ days from Cairo to the Gulf would probably prevent extreme high water and reduce the duration of flood stages.

Contiguous territory would be absolutely and permanently protected against flooding. Cairo would become a seaport in the heart of the grain section. Iron ore from Minnesota, open to year-round shipment, and coal from the Ohio Valley, meeting there, would make it the natural manufacturing center for export steel. Imported tropical fruits would be brought a thousand miles nearer their market.

No new problem in construction would be presented. Excavating the channel, facing all slopes with 8-in. reinforced concrete, and making a 25-ft sheet-pile curtain at the foot of every slope and a cut-off every mile would cost $1,364,000,000. The interest on this sum would be not more than $54,560,000 a year.

The cash offsets against this amount would be an equal average saving in actual damage from floods; elimination of the present expense of levee building and repair; and a saving of $15,000,000 a year on the present cost of freight transportation. Other advantages would be the development of swift, regular steamboat service on a safe, open channel; moving the grain belt 900 miles nearer the sea; and bringing cheap steel manufacturing to the seacoast.

It would be sound economics to have the cost made a part of the national expenditure, as are present river and harbor costs. It would also be sound economics to make the project pay for itself by a system of assessments and charges. Other countries use such a system, and the United States does so in the case of the Panama Canal.

It is true that, so far, only smaller rivers have been improved by this method, but such an improvement has never been attempted on a large scale. Nothing approaching the Panama Canal

had ever been built until the United States undertook the project. Therefore lack of precedent should not be used as an argument against the suggested flood control plan; rather should it be a challenge to our engineers to undertake it. We need river engineers comparable to our bridge and tunnel engineers, who are capable of seeing into the future and making their dreams come true.

G. H. BAYLES, M. Am. Soc. C.E.

Morgantown, W. Va.
April 6, 1934

Graphical Solution for Hydraulic Jump

TO THE EDITOR: In his article in the October issue Mr. Stevens has made a noteworthy contribution to the literature dealing with the hydraulic jump. The possibility of this phenomenon should be investigated whenever flow in an open channel at less than critical depth is retarded. If flow is to continue at a great enough depth, the jump will occur. Computation of the depth necessary to produce the jump is essential to a complete analysis of the flow conditions in the channel under consideration.

The derivation of formulas for the jump in a number of cases commonly occurring in practice is given by Mr. Stevens. With the exception of the trapezoidal and circular sections, the formulas are expressed in dimensionless terms and are presumably applicable to conduits of any size.

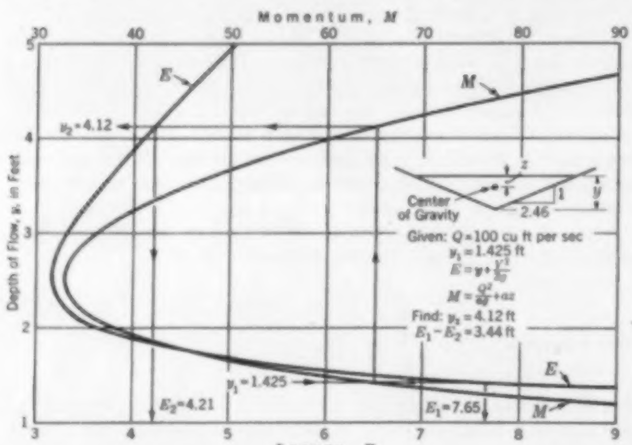


FIG. 1. MOMENTUM AND ENERGY CURVES FOR GRAPHICAL SOLUTION OF HYDRAULIC JUMP IN THE TRIANGULAR CHANNEL SHOWN

For a general solution of the hydraulic jump I prefer the graphical method outlined by Bakhmeteff in his *Hydraulics of Open Channels*, beginning page 227. Also of interest is the article, "The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures," by Julian Hinds, M. Am. Soc. C.E., in the *Engineering News-Record* for November 25, 1920, page 1034. If for a conduit of any shape, the area and depth from water surface to center of gravity of the section can be tabulated for corresponding depths of water, y, the M and E curves (momentum and energy) can be plotted for any discharge, and the downstream depth and the energy loss may be taken directly from these curves. This method of solution for Mr. Stevens's example of the triangular channel is shown in Fig. 1.

By expanding his Equation 12, collecting coefficients of like terms of J, and letting $\frac{b}{sy_1} = t$, the following equation results:

$$J^4 + \frac{(5t+2)}{2}J^3 + \frac{(3t+2)(t+1)}{2}J^2 + \left[\frac{t^2}{2} + (t-6r)(t+1)\right]J - 6r(t+1)^2 = 0 \quad . \; . \; . \; . \; . [1]$$

This is also a dimensionless equation, and is applicable to any trapezoidal channel of any size or shape. It is to be noted that the ratio, $\frac{b}{sy_1}$, is that of the area of the rectangular part of the channel to the sum of the triangular parts. If the side slopes are vertical, the channel is a rectangle, the ratio expressed by t is infinite, and my Equation 1 can be reduced to Mr. Stevens's Equation 6,

$$J^2 + J - 4r = 0 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . [2]$$

On the other hand, if the bottom width is zero, the channel is triangular, the ratio t is zero, and Equation 1 reduces to Mr. Stevens's Equation 7,

$$J^4 + J^3 + J^2 - 6r(J+1) = 0 \quad . \; . \; . \; . \; . \; . \; . [3]$$

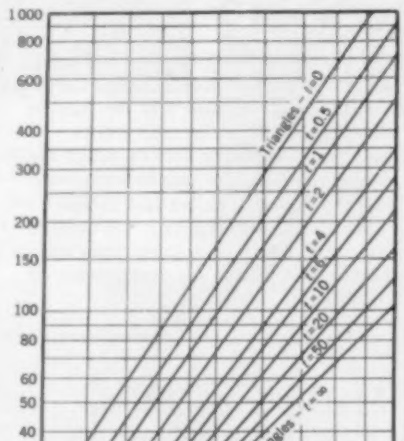
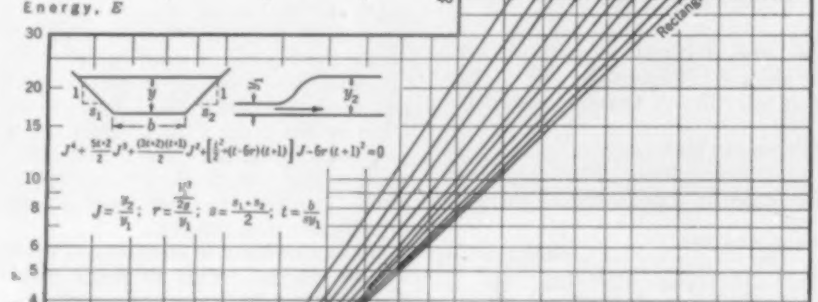



FIG. 2. CHART FOR SOLVING HYDRAULIC JUMP IN TRAPEZOIDAL CHANNELS

Equation 1 is not suitable for the solution of the problem of the hydraulic jump in individual cases. It does, however, offer a graphical solution for all trapezoidal, rectangular, and triangular sections. The relation between r, t, and J for all values of J less than 20 is shown in Fig. 2. The range of t is from zero to infinity, and it is interesting to note that all possible values of J for the trapezoidal sections are bounded on the one side by the triangular sections and on the other by the rectangles. It should be said that the curve for rectangular sections ($t = \infty$) is identical with Lindquist's curve (*Anordningar för effektiv energiomvandling vid foten av overfallsdammar*, Anniversary Volume, Royal Technical University, Stockholm, 1927; also, "Die Energieumwandlung an Wehren," *Primer Congres des Grand Barrages*, Stockholm, 1933), except for the minor fact that Lindquist plotted J against λ, where $\lambda = 2r$. His value for λ was so chosen as to be unity at the critical depth.

The use of the chart shown in Fig. 2 is simple. With known values of velocity and depth in a channel of known dimensions, compute r and t, enter the chart, and find a value of J. This determines the depth downstream, and all the other elements follow simply. Graphical calculations of the examples given by Mr. Stevens for rectangle, triangle, and trapezoid check his computed results very closely.

G. H. HICKOX, Jun. Am. Soc. C.E.
Instructor in Mechanical Engineering, University of California

Berkeley, Calif.
March 28, 1934

SOCIETY AFFAIRS

Official and Semi-Official

Plans Perfected for Vancouver Convention

Situated between the mouths of the Fraser River and Burrard Inlet, Vancouver, British Columbia, is an ideal place for the Sixty-Fourth Annual Convention of the Society. The city is nearly surrounded by water and has a background of snow-capped mountains covered with evergreen forests—a setting of great natural beauty.

Here it is that from July 11 to 14, 1934, those attending the Society's Convention will be the guests of the Engineering Institute of Canada. The two societies will meet together in joint sessions on Wednesday, July 11. A Convention program in keeping with the developments that are taking place in the Northwest has been arranged. It should prove of personal interest and professional value to all who attend.

After a welcome on behalf of the Society's hosts, the Canadian engineers, and a reply on behalf of the Society by President Eddy, an address on the relation of the Hudson Bay Company to the development of Canada will be delivered by an official of that organization. At the joint luncheon which will follow, the President of the Association of Professional Engineers of British Columbia, A. S. Gentles, is scheduled to outline the growth and work of that association.

Wednesday afternoon is to be given over to a joint technical session devoted to the development of the Columbia River drainage basin. There will be speakers from both the Canadian and the American societies. Provision is being made to develop the subject with as much discussion as time permits. In the evening, the principal address after the joint dinner will deal with the work of the Royal Engineers in British Columbia.

All day Thursday, July 12, the Technical Divisions will hold sessions, including the Power Division, the Irrigation Division, the Sanitary Engineering Division, and the Highway and Construction Divisions in joint session. Announcement of the subjects, the speakers, and the discussers will be given in the detailed program. Arrangements have been made to hold an informal dinner and dance at the Grouse Mountain Chalet on Thursday evening. This interesting spot, high on a plateau 4,000 ft above Vancouver and 18 miles away, is reached by a fine mountain highway. The view of the harbor and city at night alone is worth the trip. The vista includes Mt. Baker, a hundred miles to the south.

On Friday an all-day excursion by boat up the strait to the Powell River will provide an unequaled opportunity to view at first hand a pulp and paper mill. With an abundance of water power and forests, pulp- and paper-making constitute one of the major occupations of this region. The trip itself is replete with scenery typical of the coast of British Columbia—beautiful bays and inlets, and fine summer homes. Luncheon will be served at Powell River, and dinner on the boat. This excursion should be a never-to-be-forgotten occasion.

The final day of the Convention is to be devoted to inspection trips for those who wish to visit hydro-electric plants, the work done by the Greater Vancouver Water District, or one of the numerous local improvements. For others, opportunity to play golf on one of the many magnificent courses will be provided. Still others may wish to take trips to Seymour Canyon, Capilano Canyon, and Whytecliffe.

To reach Vancouver a number of methods and routes are available—automobile, boat, airplane, and railroad. For those from the South, East, and Middle West, a choice of scenic rail routes is available. The trip through the Canadian Rockies on the Canadian Pacific Railroad, with stop-offs at Banff and Lake Louise, will provide a magnificent choice. For those who can start early enough, arrangements are being made to include a night's stop in Victoria, on Vancouver Island, and a boat trip from there to Vancouver. For those who patronize the Great Northern Railway, a stop-off at Glacier National Park will give an opportunity to motor over the newly completed Going-to-the-Sun Highway over Logan's Pass in the Rockies, a scenic mountain highway of unsurpassed grandeur. Another attractive route utilizes the Northern Pacific Railway as far as Yakima, Wash., where a transfer to buses is possible. From Yakima connection is made with the new Sunrise Highway through Rainier National Park, to Seattle. The trip is then continued from Seattle to Vancouver by steamboat. The only western railroad to be electrified through a greater part of its length is the Chicago, Milwaukee and St. Paul, and the trip through the Rocky, the Bitterroot, and the Cascade mountains affords an intimate opportunity to see the Western mountains at their best. From the Southwest the trip can be made entirely by water along the Pacific Coast, by railroad, by paved, high-speed highways, or by airplane.

Consideration should be given to the possibility of visiting Alaska and the Yukon country by boat along the Inside Passage either before or after the Convention. Vancouver is relatively close to this most northern possession of the United States, and the passage is without the discomforts of an ocean voyage. On the return from Alaska the boat may be left at Prince Rupert and the trip continued over the Canadian National Railway by way of Jasper National Park.

© Leonard Frank

Vancouver, B.C., with the Snow-Capped Twin Peaks, the Lions, in the Background
Here the Engineering Institute of Canada Will Be the Host of the Society at Its Sixty-Fourth Annual Convention, July 11–13, 1934

Thus the Convention provides not only a motive for getting acquainted with some of the finest mountain scenery in the world, but also an occasion for making professional contacts with our Canadian neighbors and renewing friendships within the Society. Complete details of the Convention, railroad rates, and other needed information will appear in the program, to be printed in full in a subsequent issue of CIVIL ENGINEERING.

Secretary's Abstract of Executive Committee Meeting on March 23, 1934

THE Executive Committee met at the Hotel Washington, Washington, D.C., on March 23, 1934. Present were Messrs. Harrison P. Eddy, President, in the chair; George T. Seabury, Secretary; and Messrs. Crocker, Hammond, Hogan, and Lupfer.

Engineering Code

A resolution was adopted recommending and submitting for approval a revised Code of Fair Competition for the Engineering Division of the Construction Industry dated March 22, 1934, and authorizing the chairman of the Society's Committee on Code, Carlton S. Proctor, M. Am. Soc. C.E., to present this code, negotiate its development and consideration with the National Recovery Administration, to accept and approve such changes and amendments as may be desirable or necessary, and finally, to sign the Code as the Society's representative, after approval of the Code by the President of the Society.

The following members were appointed to constitute a Committee on Code and authorized to associate with themselves representatives of the American Institute of Consulting Engineers, the American Society of Mechanical Engineers, and the American Society of Heating and Ventilating Engineers: Carlton S. Proctor, Dean G. Edwards, Frank A. Marston, Frank A. Randall, and Henry J. Sherman, all Members Am. Soc. C.E.

1934 Annual Convention: Vancouver

The date for the Annual Convention to be held in Vancouver, British Columbia, was fixed as July 11–13, 1934, at the Hotel Vancouver. (The meetings of the Board of Direction will be held on Monday and Tuesday, July 9–10, 1934.)

National Planning Board

It was decided to refer to the American Engineering Council the desirability of urging the appointment of an engineer on the National Planning Board.

Construction Code Authority

Appointment was reaffirmed of John P. Hogan, a Vice-President of the Society, as a member of the Construction Code Authority.

Other Matters

Other routine matters were considered.

Ethics of Evaluation or Investigation by Engineers Personally Interested in Commercial Outcome

FOR THE ENLIGHTENMENT of all members, an abstract of suppositional questions of ethics and their proper solution, as submitted by the Society's Committee on Professional Conduct, is printed here as a guide in applying the principles in similar instances. The answer to these questions was approved by the Board of Direction at its January meeting.

Questions:

A. Is it ethical for a concern interested directly or indirectly in some particular type of installation, to solicit engagement on a free basis as investigator of utility services and rates and then to recommend its particular installation?

B. Would this be ethical if the basis were a reasonable fee, with provision for remitting the fee in case the particular installation is used?

C. After soliciting and obtaining engagement of valuation or recommendation as to utility services and rates, is it ethical for an engineer to recommend building a municipal plant using equipment in the sale of which he is directly interested?

Answer:

All three questions involve a practice that is highly unethical. Engineering services of a proper kind and well performed are valuable and must be so recognized. Such services should be adequately paid for. To offer services to a community free in the expectation that a sale of machinery and equipment will result, or to make a charge for such services and then rebate the charge after the sale are equally pernicious. No engineer, practicing as consultant, should have any interest in a concern manufacturing equipment recommended or specified by him.

These comments do not refer to properly conducted sales work by engineers. That is a legitimate field for engineering effort.

Additional Memoirs Available to Members and Friends on Request

A CONSIDERABLE number of memoirs have recently become available to supplement the list published in the September 1933 issue. These biographies represent a great deal of work on the part of interested friends and associates of the deceased. The present printing is in pamphlet form preliminary to inclusion in the first available issue of TRANSACTIONS, in which they will be distributed to all members.

Sufficient preprint copies are available for presentation on request to any interested members or friends. Requests may be addressed to the Secretary at Headquarters. The memoirs now available, not hitherto announced, include the following:

Frank George Baum	Francisco Xavier Memije
John Crichton Baxter	Frank Pierce Meserve, Jr.
Frederick Kellogg Blue	Sir Ernest William Moir
Clarence Henry Bowman	Jerome Newman
Theodore Brand	Fred Adolph Noetzli
Edward Michael Brennan	Dwight Raymond Redman
Elga Ross Chamblin	Richard Franklin Rey
Alexander Samuel Diven, 3d	James Horner Rice
Charles Steward Donald	Royden Karl Schlafly
Abraham Fairbanks Doremus	William Charles Schnabel
Abraham Leonard Drabkin	Robert Harris Simpson
Frederick James Easterbrook	Ray Hamilton Skelton
Harry Franklin Flynn	John Rodolph Slattery
Arthur DeWint Foote	David Sloan
John Peden Gardiner	Acheson Smith
Samuel Joseph Garges	Robert Charles Strachan
Carl Gayler	Holger Struckmann
Andrew Valerian Greaves	Frank Mayhew Talbot
John Alexander Griffin	Russell Thayer
Edward Buckingham Guthrie	Orville Hickman Browning Turner
	Charles Oscar Vandevanter
William Henry Hall	Hans August Evald von Schon
George Halverson	Leland Ross Walker
Thomas Jefferson Hayden, Jr.	William Richardson Webster
Harvey Sydney Henning	Charles Hunter West
Charles Edwin Jenkins	Harry Roberts Wheeler
Frank Hall Joyner	Ivan Forrest White
Eugene Ashbel Landon	Frederick Wilcock
Olin Henry Landreth	Henry Harrison Wilson
Herbert Lawrence Luther	Rollen Joe Windrow
Alfred Fellows Masury	Louis Peter Wolff
Oliver Earle Young	

Making Government Publications Useful to Engineers

IN 1931 the Society, with the intimate assistance of the heads of various Government bureaus, published Manual 7 entitled "Government Services Available to Civil Engineers." Essentially a "clew" book, this Manual of engineering practice will continue to

serve its purpose for several years, despite possible modifications in the organization of the various Government departments. The Manual has been well received, and there is evidence that it is being put to profitable use. It is designed to give the civil engineer an idea as to the Government department to which he should turn for assistance in the solution of a particular problem.

The Government official addressed may refer an inquirer to available publications in which the answer to such a problem is published in full, or he may offer other advice. The Manual serves the purpose of putting the inquirer in touch with the one who is qualified to answer his particular question. It should be noted, however, that Government bureaus cannot supply publications if the inquirer refers only to Manual 7, because there is no official connection between the Government bureaus and the Society.

If a list of Government publications useful to civil engineers is desired, a request can be made directly to the Superintendent of Public Documents for one of the following price lists:

Price List 15. U. S. Geological Survey
Price List 18. Engineering and surveying
Price List 20. Public domain
Price List 25. Transportation and the Panama Canal
Price List 36. Government periodicals for which subscriptions are taken
Price List 42. Irrigation, drainage, water power
Price List 43. Forestry
Price List 45. Roads
Price List 48. Weather, astronomy, and meteorology
Price List 51. Health (sanitation, water pollution)
Price List 53. Maps
Price List 58. Mines
Price List 59. Interstate Commerce Commission publications
Price List 64. Standards of weights and measures
Price List 70. Census
Price List 73. Handy books (books for ready reference covering many topics)

There is a general pamphlet, "Information Governing Distribution of Government Publications and Price Lists for the Office of Superintendent of Documents, Washington, D.C." Also a "Monthly Catalogue of United States Public Documents" is available at a cost of 10 cents the copy, or of 75 cents per year. In addition, most technical and public libraries will have the cumulative catalogue of public documents, in whole or in part. A catalogue of scientific publications from foreign countries is available for reference at the Library of Congress.

The Superintendent of Public Documents is required to exact a nominal charge for the publications he distributes, but there is no charge for price lists. Often the department that produces a certain publication has a limited supply available for free distribution to those who apply direct.

It is hoped that these comments will make Manual 7 more useful to members desiring information which the Government may provide.

Suggestions for Washington Award

ONE of the outstanding American honors given to engineers is the Washington award, granted annually in the form of a bronze medal or other work of art "as an honor conferred upon a brother engineer by his fellow engineers on account of accomplishments which preeminently promote the happiness, comfort, and well-being of humanity." Founded in 1915 by John W. Alvord, M. Am. Soc. C.E., it has been given annually since 1919, when the first award was made to Herbert Hoover, Hon. M. Am. Soc. C.E. Although administered by the Western Society of Engineers, the American Society of Civil Engineers has two representatives on the Commission of Award.

Up to the present time, this commission has gathered each year its own list of eligibles from which it has selected its final choice. It now has in mind a change in this procedure whereby this list of eligibles may be increased. To this end suggestions are invited from any interested member. Such suggestion may well include brief notes of accomplishment upon which the recommendation is based. It may be transmitted direct to Edgar S. Nethercut, Secretary of the Western Society of Engineers, at its headquarters, 205 West Wacker Drive, Chicago, Ill.

Projects of American Standards Association

TO FILL the need for a clearing house for nationally approved standards, the American Standards Association was organized in 1918 by the four Founder Societies and the American Society for Testing Materials. The purpose of the organization is to provide dimensional standardization so as to permit the interchange of parts and to secure general standardization of supplies and apparatus; to provide standard specifications for materials and methods of test; to define engineering symbols and technical terms used in industry; and to secure uniformity in the requirements of safety codes so that safety devices for machines and equipment may be uniform. In 1929 the American Standards Association became a member of the International Standards Association, a body which has headquarters in Basle, Switzerland, and acts as a clearing house for international cooperation in standardization.

The status of current civil engineering projects under the American Standards Association procedure, as of February 1, 1934, is given in the following paragraphs, briefed from *Industrial Standardization and Commercial Standards Monthly*, published by the American Standards Association with the cooperation of the National Bureau of Standards:

A1—Specifications for Portland Cement. A standard method for sampling and testing portland cement has been approved.

A2—Fire Tests of Building Construction and Materials. A revised standard for such tests has been submitted for approval as an American Standard.

A6—Specifications for Drain Tile. These are under preparation.

A21—Specifications for Cast-Iron Pipe and Special Castings. The report of a committee on tests completed at the Massachusetts Institute of Technology on corrosion of cast iron is to be published soon. Another committee is working out recommendations for accelerated tests for organic coatings of pipe. The recommendation of a committee on inorganic lining for a minimum thickness of $^1/_4$ in. for all sizes of pipe is under consideration. Three cold asphalt paints have been recommended for bituminous seal coats for concrete linings.

A35—Manhole Frames and Covers. The proposed American Standard is being revised as a result of comments and suggestions received up to July 1933.

A36—Rating of Rivers. Six definite recommendations for the rating of the water power of rivers, which were accepted by the World Power Conference, are to be considered for adoption as American Standards.

A37—Methods of Testing Road and Paving Materials. No new standard methods of testing were submitted during 1933.

A38—Steel Spiral Rods for Concrete Reinforcement. The text of the revised standard presents data on recommended sizes of reinforcing rods.

A45—Method of Test for Sieve Analysis of Aggregate for Concrete. Revised specifications will be submitted shortly for approval as American Standards.

A48—Forms for Concrete Joist Construction Floors. The provisions of the standard for such forms cover the main dimensions of removable and permanent forms, domes or pans of wood, steel, or other material used in concrete ribbed floor construction.

A plan of cooperation between the Government Bureau of Standards and the American Standards Association has been worked out in recent months. Standards having the full approval of both will be issued either as American Simplified Practice or as American Commercial Standards.

SIXTY-FOURTH ANNUAL CONVENTION of the Society, July 11–13, 1934, in Vancouver, British Columbia

Basis for Accrediting Engineering Colleges

ONE of the primary committees of the Engineers' Council for Professional Development (E.C.P.D.), as previously noted in these pages, is charged with the study of engineering schools, including the standards upon which they may be duly accredited. At its meeting in January 1934, the Board of Direction of the Society approved the recommendation of the E.C.P.D., contained in its first annual report, that it be authorized to act as an accrediting agency for engineering schools. According to this report, the E.C.P.D.'s Committee on Engineering Schools felt it unwise to recommend minimum hours and credits as a preliminary to accrediting. It believed that "there should be no published prescription regarding curricula."

The committee did suggest, however, a general "Basis for Accrediting Engineering Colleges," in the following topical form:

I. Purpose of accrediting shall be to identify those institutions which offer professional curricula in engineering worthy of recognition as such.

II. Accrediting shall apply only to those curricula which lead to degrees.

III. Both undergraduate and graduate curricula shall be accredited.

IV. Curricula in each institution shall be accredited individually. For this purpose, the E.C.P.D. will recognize the six major curricula: Chemical, Civil, Electrical, Mechanical, Metallurgical, and Mining Engineering—represented in its own organization, and such other curricula as are warranted by the educational and industrial conditions pertaining to them.

V. Curricula shall be accredited on the basis of both qualitative and quantitative criteria.

VI. Qualitative criteria shall be evaluated through visits of inspection by a committee or committees of qualified individuals representing the E.C.P.D. The visits of inspection either as to entire institutions or as to specific curricula may be waived at the discretion of the Council.

VII. Quantitative criteria shall be evaluated through data secured from catalogs and other publications, and from questionnaires.

VIII. Qualitative criteria shall include the following:

1. Qualifications, experience, intellectual interests, attainments, and professional productivity of members of the faculty.
2. Standards and quality of instruction:
   a) In the engineering departments.
   b) In the scientific and other cooperating departments in which engineering students receive instruction.
3. Scholastic work of students.
4. Records of graduates both in graduate study and in practice.
5. Attitude and policy of administration toward its engineering division and toward teaching, research, and scholarly production.

IX. Quantitative criteria shall include the following:

1. Auspices, control, and organization of the institution and of the engineering division.
2. Curricula offered and degrees conferred.
3. Age of the institution and of the individual curricula.
4. Basis of and requirements for admission of students.
5. Number enrolled:
   a) In the engineering college or division as a whole.
   b) In the individual curricula.
6. Graduation requirements.
7. Teaching staff and teaching loads.
8. Physical facilities. The educational plant devoted to engineering education.
9. Finances; investments, expenditures, sources of income.

Progress of Engineering Code

LAST MONTH in these columns I indicated the intended scope to be embraced in the Engineering Code of Fair Practice. There was also announcement of the personnel of the Committee on Code established by the Executive Committee of the Society.

The formation of this committee was a most helpful move. Mr. Marston is particularly well acquainted with the work of sanitary engineers; Mr. Randall with that of structural engineers; Mr. Sherman with that of those engaged on waterways, shore protection, and beach erosion; and the writer with foundation and subaqueous problems. The assistance rendered by Gavin Hadden, M.Am. Soc. C.E., representative of the American Institute of Consulting Engineers, interested in the design of stadiums, airports, and the like; that of Henry Meyers, representative of the American Society of Mechanical Engineers, acquainted with mechanical installations; and that of John G. Eadie, representative of the American Society of Heating and Ventilating Engineers, acquainted with heating and ventilating systems and the like, as incorporated in buildings, has enabled the group to have a wide and comprehensive view of the problems. Mr. Edwards gave his attention to contact with NRA officials.

This Code group met on March 29 and devoted 13 hours that day to the study of the draft of the Code approved by the Executive Committee of the Society as a basis for consideration. Copies of that draft have been sent to the presidents and secretaries of all of the Society's 56 Local Sections, to every other organization of engineers that was thought to have an interest in the matter, and to as many individuals as had expressed an interest in the Code. Comments were asked for, preferably in the form of exact phraseology, and many such comments were received.

These comments and suggestions in many instances were extremely helpful. They were all reviewed by the Code group at a two-day meeting held on April 12 and 13, and many of the suggestions led to changed phraseology considered to result in material improvements.

At this present moment it may be said that there is proposed no considerable change from the draft of March 22. Its major characteristics remain, although some of the details have been modified in the light of the suggestions received and, in general, the present draft is felt to be the most satisfactory of any draft so far developed. Informal contacts with the NRA officials are being continued and further suggestions from all those interested are greatly to be desired in order that the Code group when it shall meet again, on approximately the first of May, may have the benefit of as wide an expression of opinion as possible.

There is no question but that the Society has done well to actively assume leadership in this matter of the Code by providing the facilities for a widespread expression of opinion from those to be affected by it.

CARLTON S. PROCTOR, M. Am. Soc. C.E.
Chairman, Committee on Code

Columbia University Scholarship Available

THROUGH the courtesy of Columbia University, New York, N.Y., the Society has at its disposal a scholarship in civil engineering at that institution. This scholarship was established in honor of Horatio Allen, a Columbia graduate of the class of 1823 and the fifth President of the Society.

A candidate for this scholarship must have completed at least two years of general preparatory work in an accredited engineering school. The award will be made by the Scholarship Committee of the Society to a candidate whose record meets with the approval of the university authorities. Depending on the class with which the successful candidate enters and his progress during the year, the scholarship may be held by him from one to three years.

For the year beginning in September 1934, the scholarship covers tuition only, which however includes all university fees. It has a total value of about $400. Applications should be addressed to the Secretary of the Society and must be in hand by June 1. More detailed information will be given on request.

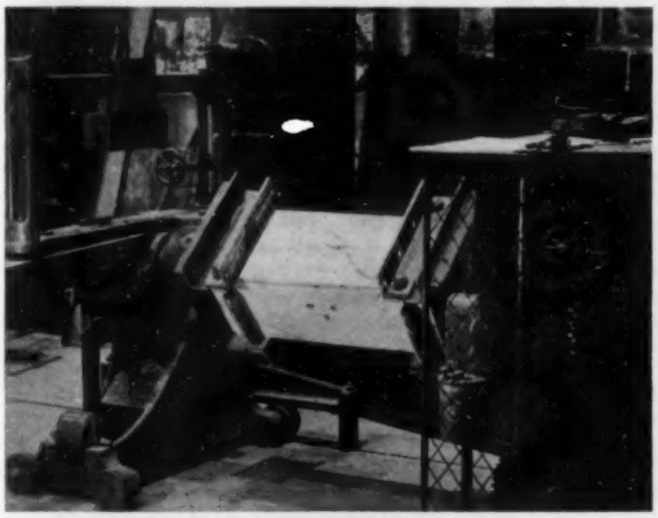
A Preview of Proceedings

In May will appear the last number of Proceedings *before the summer interval of two months during which publication is suspended. The next succeeding, or August number, will be mailed on August 15. The summer interval provides time for preparation of the next volume of* Transactions.

In the May number of Proceedings *more discussions than usual will be included, among them several interesting closures. Four new papers are in preparation for this issue: one on stresses in structures, one on torsional strength of concrete, one on flood control reservoirs, and the last, on the pressures exerted by sea waves.*

Stresses in Space Structures

Domes and other forms of space structures have been freely used in Europe for many years but are relatively uncommon in this country. When met with at all here, they are usually in the form of a tower intended for transmission or signal purposes, or for the support of tanks or other structures. Probably dome-like structures would more frequently be employed in architectural design if the analysis of the stresses were not so tedious and difficult to make. The chief difficulty lies in the three-dimensional character of the problem. In a paper, "Introduction à la statique graphique des systèmes de l'espace," published in 1926, Prof. Benjamin Mayor, of the University of Lausanne, pointed the way to a simplification of the problem by showing that all space forces may be replaced by corresponding systems of co-planar forces, and that a study of the equilibrium of the latter leads at once to the desired spatial stresses. His method, however, is based on a knowledge of the linear complex, a field of modern mathematics not generally understood by engineers.



Laboratory Test of Concrete in Torsion
Specimen Shows Typical Failure by Diagonal Tension

In the paper by Prof. F. H. Constant, Professor of Civil Engineering at Princeton University, entitled "Stresses in Space Structures," to appear in the May number of Proceedings, the subject has been developed in a simple and understandable manner, employing only well-known principles of three dimensional mechanics. For every set of concurrent forces meeting at a joint in a space structure, there is a corresponding set of non-concurrent forces, called conjugate forces, which may be made to lie in any conveniently chosen reference plane. These latter forces are also in equilibrium, and their solution may be effected in the usual manner by the use of the three static equations of equilibrium applied either analytically or graphically. The conjugate vectors are easily and quickly located, and the simplification of the problem, by reducing it from a three-dimensional to a two-dimensional character, is at once apparent.

The theoretical treatment of the subject is followed by an application of the method to the solution of a four-legged tower, which, although as simple an example as could be chosen, completely illustrates all the points likely to arise in any more complex structure.

Six Drag-Line Excavators at Work on Pilot Cut for Willow Point Cut-Off
Mississippi River, near Vicksburg, Miss.

Experiments with Concrete in Torsion

During the development of the design of reinforced concrete structures, the effect on individual members of torsional stresses has received but little attention. The adaptability of reinforced concrete to almost any kind of building work has resulted in some structural forms in which torsional stresses do occur; these stresses may either be due to transmission of twisting moments incurred by eccentrically placed loads or they may be a result of deflections of adjacent parts of the structure, as in marginal beams. The paper by Paul Andersen, Assoc. M. Am. Soc. C.E., explains the behavior under torsion of plain and reinforced concrete and presents the results of the testing to destruction of 48 specimens unreinforced or reinforced in various ways. The experiments were carried out in the Materials Testing Laboratory of the University of Illinois.

As concrete in torsion always fails in diagonal tension, it follows that the most effective means of raising its ultimate strength is spiral reinforcement at 45 deg to the axis of twist. Completely closed ties normal to this axis, however, will also provide additional strength. An analysis of the test data indicates that, if spiral reinforcement is used, the concrete and the steel can be assumed to share the work of shear resistance in a manner similar to conventional design practice of beams in bending.

The Reservoir as a Flood Control Structure

A detailed outline of the procedure of conducting a study of reservoirs for flood control is presented in the paper by George R. Clemens, Assoc. M. Am. Soc. C.E. It is at once so simple in its outline and so complete in its scope that it should serve as a guide

Dynamometer Crib to Measure Wave Pressures
Eastern Sea Wall, Toronto Harbor Improvement, 1915

for students and as a reference work for practitioners in this field. Admittedly, a considerable part of the paper is a compilation of information that is well known to most flood control engineers. Nevertheless, it contains many new slants. It is hoped that such discussion will ensue as will crystallize in the mind of the reader his own personal logic for application in similar cases.

After a rather complete introduction, including the basic definitions, Mr. Clemens outlines the preliminary investigations required to determine the nature of structures to be incorporated in any flood control plan. This is followed by a description of a design of a reservoir system including flood routing for specific cases and a detailed analysis of valley storage effects. There is then presented a discussion of the combined use of reservoirs for flood control and other purposes, such as water power, irrigation, navigation, and water supply. Finally, Mr. Clemens presents a section on reservoirs combined with other structures, such as side channels, floodways, channel improvements, and levees. Practical examples are included to demonstrate cardinal points in the argument, and these points are supported by data actually observed in the field. The paper should serve as an excellent framework upon which to build a standard practice in such matters.

Wave Pressures on Sea Walls and Breakwaters

In his paper, David A. Molitor, M. Am. Soc. C.E., has made a contribution to engineering literature on a subject on which comparatively little useful information is available. The evaluation of the probable wave dimensions for a certain site, exposure, and maximum wind velocity is presented here for the first time. This should enable the design of breakwaters with sufficient exactness to ensure safety, yet without resort to excessive wastefulness.

When the safety of the Toronto Harbor breakwater design was challenged by the contractor, before its actual construction was undertaken in 1915, it became necessary to establish proof of adequate design. In the absence of any authenticated method of procedure it was necessary to devise a method of approximate appraisal of the probable wave force for the particular exposure and depth of water, and then show by comparison that the same method of analysis, when applied to numerous existing structures, gave results which agreed well with their actual behavior after they had weathered many severe storms. It was thus a case of first inventing a method of analysis and then proving the method to be reliable.

After the claim of inadequate design was refuted by this method of approach, a thorough investigation was made by a board of consulting engineers, with the result that the structures were built without increased dimensions. They have now weathered all storms for the past 18 years without showing any signs of weakness. It is expected that discussions of this paper will add some valuable information and thus render a distinct service to the profession.

News of Local Sections

Central Ohio Section

Various business matters were discussed at a meeting of the Central Ohio Section, held in Columbus on February 15. The speaker of the occasion was Robert Devereaux, of the U. S. Bureau of Public Roads, who discussed the work of the bureau and the state highway program for Ohio.

There were 36 present at the meeting of the Central Ohio Section held in Columbus on March 15. After discussion of several business matters, the members were privileged to hear Robert M. Miller, of Cincinnati, who spoke on the topic, "Engineering Which Recognizes Environment." A progress report of the Engineers' Code Committee was also heard.

Chattanooga Section

A regular meeting of the Chattanooga Section was called to order on March 12. The feature of the occasion was the annual election of officers, which resulted as follows: Warren R. King, President; W. H. Wilson, First Vice-President; G. R. Kavanagh, Second Vice-President; and A. W. Crouch, Secretary-Treasurer. Numerous business matters were also taken care of at this session.

Cincinnati Section

At the meeting of the Cincinnati Section, held at the Engineers Club on March 5, the speaker was L. A. Boulay, Ohio State Engineer for the Public Works Administration. In his talk Mr. Boulay reviewed conditions that gave rise to the creation of the Public Works Administration and presented many facts and figures concerning its operation since the opening of his office last September. There were 60 members and guests in attendance.

Cleveland Section

There were 68 in attendance at a meeting of the Cleveland Section held on January 10. The guests of the Section included Student Chapter members of Akron University and of the Case School of Applied Science as well as ten former presidents of the Section. After a business session the meeting was turned over to Alonzo J. Hammond, Past-President of the Society, who spoke on the subject of the activities of the Society during the past year.

Business matters occupied part of the session of the meeting of the Cleveland Section held on April 3, with 28 members and guests present. The speaker of the occasion was L. C. West, Finance Director of the City of Cleveland, who described the financial difficulties of the city. An animated discussion from the floor followed Mr. West's talk.

Colorado Section

Through the courtesy of the Board of Water Commissioners of the City of Denver, a symposium on water problems of local interest was presented at a meeting of the Colorado Section held in Denver on January 15. This interesting collection of papers provoked considerable discussion. There were 48 present. The Denver Athletic Club was the scene of a dinner meeting of the Section on February 19. The functions of the city engineer's office were described by Charles Davis, Sanitary Engineer of the City of Denver. An exchange of reminiscences on the subject and an interesting discussion of it from the floor followed this talk. There were 27 members and guests in attendance.

Dayton Section

The February meeting of the Dayton Section was held at the Engineers Club on the 26th. Reports of various committees were presented and approved, after which George E. Shafer, engineer of tests for the Armco Culvert Manufacturers' Association, was introduced. Mr. Shafer gave an illustrated talk on the subject, "Design of Small Drainage Structures." There were 22 present at the meeting, including guests from the University of Dayton Student Chapter.

Ithaca Section

A dinner meeting of the Ithaca Section was held on January 22 on the campus of Cornell University. The guest speaker was Elwyn E. Seelye, of New York, N.Y., who spoke on "Mistakes in Building Design." There were 38 present. On February 16 a dinner meeting was held in Ithaca. After the meeting the group adjourned to the Baker Auditorium at Cornell University to listen to Col. Hugh Cooper, who was a guest of the university. Colonel

Members of the Ithaca Section and of the Cornell University Student Chapter Inspecting the Erie Railroad's Grade Separation Project

Cooper lectured on the subject, "The Dnieper Dam and Some Observations Regarding the Russian Experiment."

On March 15 members of the Section and of the Cornell University Student Chapter enjoyed an inspection trip to the Erie Railroad's grade separation project at Elmira, N.Y. Officials of the Erie Railroad and of the New York State Department of Public Works conducted the 80 members of the Section and Student Chapter who made the trip over the five-mile project. In the evening the Section held a dinner meeting at which A. M. Knowles, Structural Engineer of the Erie Railroad, was speaker. There were 65 present.

Kansas City Section

On March 16 a joint meeting of the Kansas City Section and local branches of the American Institute of Electrical Engineers, the American Society of Mechanical Engineers, and the American Society of Heating and Ventilating Engineers was held. The principal speaker was Prof. J. B. Whitehead, dean of the School of Engineering, Johns Hopkins University, who spoke on the subject of "Liquid Dialectrics." Before the general meeting convened, the Kansas City Section attended a dinner meeting, at which various business matters of the Section were transacted.

Kansas State Section

The Kansan Hotel in Topeka was the scene of the luncheon meeting of the Kansas State Section held on March 17. Several business matters were discussed, after which LaMotte Grover presented a paper entitled "Arc Welding in Bridge Construction." There were 26 present.

Los Angeles Section

The Los Angeles Section met at the University Club on March 14. After a business session, the program of the evening was opened with the introduction of Palmer Conner, assistant secretary of the Title Insurance and Trust Company, who spoke on the history of Spanish grants in California. The chief speaker was A. H. Ayers, Chief Engineer of the Six Companies Inc., who discussed the subject, "Boulder Dam from the Construction Standpoint." In connection with his talk four reels of motion pictures, prepared jointly by the Government and the contractors, were run to show the development of the project to date. At the conclusion of Mr. Ayers' remarks the meeting was thrown open to general discussion. There were 212 at the dinner preceding the meeting and more than that number at the meeting itself.

Maryland Section

At a meeting held in Baltimore on January 25, the Maryland Section was addressed by Harold West, chairman of the Public Service Commission of Maryland, who spoke on the subject, "Problems of Public Utilities Regulation." An interesting discussion from the floor followed. On March 1 the Section held another meeting, at which the members were privileged to hear Dr. Broadus Mitchell, of the School of Business Economics of the Johns Hopkins University, who commented on the current American economic scene.

Metropolitan Section

At the regular meeting of the Metropolitan Section, held in New York, N.Y., on April 18, the subject of "Destructive Earthquakes—Their Probable Frequency and Magnitude in This Region" was presented by the Rev. Joseph J. Lynch, S.J., head of the Department of Physics of Fordham University. He covered general phases of earthquake occurrence, magnitude, and detection, together with the instruments used in This work, especially dwelling on the probabilities as affecting New York City. An interesting discussion followed. At the business meeting of the Section the nominating committee for officers for the ensuing year was appointed. Refreshments were served. The attendance was about 250.

Northwestern Section

The Northwestern Section held a meeting at the University of Minnesota on March 1. The session was given over to a discussion of the newly developed processes of placing concrete. The principal speaker was E. W. Seemann, supervising engineer for the Merritt, Chapman and Whitney Corporation, of Chicago, Ill. Moving pictures illustrating the pumpcrete method of placing concrete were shown by J. Harry Pinson, of the Chain Belt Company, of Milwaukee, Wis. There were approximately 60 in attendance at the meeting.

Pittsburgh Section

Officers elected by the Pittsburgh Section for 1934 are as follows: E. N. Hunting, President, and Nathan Schein, Secretary.

Philadelphia Section

A spirit of merrymaking prevailed at the tenth annual social dinner and meeting of the Philadelphia Section, held at the Engineers Club on February 17. Singing, card games, and dancing were features of the interesting and varied program of entertainment. The guest of honor of the occasion was George T. Seabury, Secretary of the Society.

Portland (Ore.) Section

There were approximately 110 present at a meeting of the Portland (Ore.) Section held in the Public Service Building on February 16. The first speaker was Floyd Allen, a former president of the Professional Engineers of Oregon, who gave a brief description of the work that he has done in teaching the elements of engineering and surveying to Boy Scouts. He was followed on the program by Lewis A. McArthur, vice-president and general manager of the Pacific Power and Light Company, who gave an informative, illustrated talk on the subject, "Mapping Oregon—Past, Present, and Future." A general discussion from the floor followed his address.

San Diego Section

On February 27 a dinner meeting of the San Diego Section was held at the Churchill Hotel. A talk was given by H. A. Cordes, of the General Electric Company, who spoke on the subject, "The World's Largest Automatically Operated Hydro-Electric Development." The automatic features of the plant in question, which is on the Ohio River at Louisville, Ky., were illustrated by a reel of motion pictures. Also, a brief talk on the general characteristics of the Ohio River and on the structural details of the plant was given by H. A. Noble, president of the Section.

Student Chapter News

Drexel Institute

The Drexel Institute Student Chapter sponsored a meeting of the six Student Chapters in the Philadelphia district on February 8. The first speaker was Allen P. Richmond, Jr., assistant to the Secretary of the Society, who commented briefly on the recent organization of Student Chapters in the New York City district. Then Willard T. Chevalier, publishing-director of the McGraw-Hill Publications Company, of New York, N.Y., gave an interesting talk on the position of the engineer in the present economic order.

Ohio Northern University

On February 15 the Ohio Northern University Student Chapter celebrated civil engineering day by holding a meeting, which was attended by 80 members of the faculty and student body. An interesting address on "Fire Protection Engineering," was given by Edmund J. Miessler, superintendent of the Ohio Inspection Bureau, of Lima, Ohio.

Tulane University

During the past few months the Tulane University Student Chapter enjoyed two inspection trips to points of engineering interest. On December 21, 1933, a visit was made to the mill of the Lone Star Cement Company of Louisiana, which is located at New Orleans. On January 9, 1934, members of the Chapter availed themselves of the privilege of visiting the repair shops and yards of the Southern Pacific Railroad Company, at Algiers, La., across the river from New Orleans.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for June

The Metropolitan District of the Commonwealth of Massachusetts is the fourth largest district in the country. When in 1918 the existing water supply system began to be inadequate for the needs of Boston, house meters were installed. Before this expedient failed, additional water began to be developed on the Ware and Swift rivers. Two large earth dams will form the Quabbin Reservoir, and a 25-mile pressure tunnel will connect it with the present Wachusett Reservoir. This development is explained in an article prepared by the Chief Engineer of the Metropolitan District Water Supply Commission, Frank E. Winsor, M. Am. Soc. C.E., scheduled for June.

Los Angeles disposes of its sewage in the Pacific Ocean by an outfall discharging at a point a mile offshore. The 130 mgd of raw sewage undergoes no treatment other than passage through bar and revolving cylindrical screens. As available space for the burial of screenings became restricted, steps were taken in 1932 to incinerate them. A description of the equipment built for this purpose and of the tests run on the destructor before it was accepted by the city is given by H. G. Smith, Engineer of Sewer Design, in another article prepared for June.

Construction of the Madden Dam, being built by the Government to augment the water supply for the Panama Canal locks, was begun early in 1932. In an article by Irwin E. Burks, Chief Concrete Technician on the work, prepared from a talk before the Panama Section, the methods for designing the concrete mixes and controlling the output of the mixing plant are made clear.

Another article scheduled for June deals with the history of the Maryland-West Virginia boundary controversy. The author, Lynn Perry, M. Am. Soc. C.E., has prepared a comprehensive review, beginning with 1632, when the original charter for Maryland was issued by Charles I, and ending with 1912, when the matter was finally settled by the U. S. Supreme Court. This clear presentation of a confusing boundary difficulty is of considerable general, as well as engineering interest.

Some phases of the construction of the Independent Subway System in New York, N.Y., have already been described in Civil Engineering. In an article to appear in the June issue, Alfred Brahdy, M. Am. Soc. C.E., presents some additional phases of the work, particularly the $2^1/_2$ miles along St. Nicholas Avenue, which cost \$20,000,000, and the two miles under the Grand Concourse, which cost \$12,000,000 more.

"Vocational Guidance in Engineering Lines"—An Appreciation

By Frederic Bass, M. Am. Soc. C.E.

Professor of Municipal and Sanitary Engineering, University of Minnesota, Minneapolis

During recent years many means have been provided for apprising the inquiring boy or young man about to choose a career, of the possibilities and limitations of the engineering profession. Outstanding among this literature is the volume here described, sponsored by J. A. L. Waddell, M. Am. Soc. C.E., and issued under the aegis of the American Association of Engineers. Because it is perhaps the most ambitious advisory volume so far printed and because it represents an altruistic effort by a national organization of engineers, this notice is somewhat more lengthy than usual. If "Vocational Guidance in Engineering Lines" is not available in the local library, members may obtain copies from the publisher, the Mack Printing Company, Easton, Pa., at the rate of \$2.50 per copy postpaid.

Probably the volume, *Vocational Guidance in Engineering Lines*, is more completely descriptive of professional engineering than any other publication. Although it was compiled and edited primarily as a guide to young men, it also is widely informative to engineers, to engineering teachers, and to the public. It is an illustrated book of about 550 pages, containing ten general chapters plus fifty chapters written by as many leading practicing engineers, each of whom describes the work included in his specialty and the requirements of the workers in that field. The various fields are considered not only in their detailed technical aspects, but also in their relations to one another, and to some extent in their relations to economic and social objectives. The authors appeal to the idealism of youth and to the experience of maturity. The element of romance is brought into direct contact with the sobering facts of work and discipline required for successful effort.

Most of the authors assume the desirability of a formal technical education as a prerequisite for success in engineering practice, although notable exceptions to this rule are recognized. The prospective engineering student is warned that he must like mathematics, hard study, and strenuous physical work; that he must not object to discipline; that he must exceed minimum requirements; and that he must be ambitious to excel in his work rather than to accumulate wealth. To this might be added the warning that less than 40 out of 100 students beginning an engineering course finish it and less than 30 finish it in the specified time. Seventy per cent of the failures are due to poor preparation, lack of ability, and lack of interest. Of graduates, less than 40 per cent remain in technical engineering employment. In addition to these facts, there is stressed the ever-present requirement of professional work—that of responsibility. The increasing demand on the engineer's sense of responsibility is implied and even emphasized in many chapters.

As a whole the book may be said to contain substantially all the information that a young man intending to follow the engineering profession could hope to find. Its style is uniformly direct, lucid, and compact. Written in the spirit of encouragement to qualified young men, it should exercise an influence in elevating and broadening the engineering profession.

Exhibits of Structural and Hydraulic Models

Immediately preceding the annual meeting of the Society for the Promotion of Engineering Education, to be held at Cornell University in June 1934, there will be a $2^1/_2$-day conference, June 19–21, on the use of models in structural and hydraulic engineering, as well as in some other branches. In connection with this conference, arrangements are being made for an exhibition of models from all over the United States and Canada.

The committee in charge of this project will greatly appreciate the cooperation of everyone who is willing to loan such models or apparatus for the duration of the exhibit, or who will notify them of possible sources whence such models may be secured. In cases where the structures themselves cannot be sent, pictures and cuts will be gratefully received. Dean D. S. Kimball of Cornell will provide safe display facilities and see that the articles are returned promptly to the owners at the close of the exhibit.

Information and suggestions should be addressed to George E. Large, Assoc. M. Am. Soc. C.E., in the case of structural

models, and to R. W. Powell, M. Am. Soc. C.E., in the case of hydraulic models, both of the College of Engineering, Ohio State University, Columbus, Ohio.

Student Bridge Design Competition

SIXTY-TWO students, representing 24 colleges in the United States, participated in the Sixth Annual Students' Bridge Design Competition of the American Institute of Steel Construction. A jury of nationally known engineers and architects selected the ten best from the preliminary drawings for entry into the final judging to be held on May 3. All the entrants in the competition are actively engaged in studying engineering or architecture.

The ten selected to submit final renderings are: K. R. Darrah, Robert A. Jones, J. F. Nowak, George Pistey and Henry C. Staeger, of Rensselaer Polytechnic Institute, Troy, N.Y.; Harry E. Rodman and Robert J. Sharp, of Iowa State College; Albert R. Nozaki and Harold E. Steinberg, of the University of Illinois; and David Hiat, of New York University.

From these ten, on May 3, the jury will select a prize winner who will be awarded $100 in cash. There will be a second prize of $50, and certificates of merit will be given those winning third, fourth, and fifth places.

The problem is the design of a steel tower for a small highway suspension bridge in which the main span is 500 ft; the cable sag at mid-span, 60 ft; side spans, each 240 ft from center of towers to anchorage connections; clear roadway width, 30 ft; elevation of the under side of roadway at towers, 50 ft above water level; and top of masonry piers, 10 ft above water level.

Request for Data on Thomas Telford

IN CONNECTION with the centenary of the death of Thomas Telford, first president of the Institution of Civil Engineers, which will be observed this year, Sir Alexander Gibb, M. Am. Soc. C.E., is writing the life of this noted Scotch engineer. Believing that there are various items of Telfordiana in this country, which would help him in the preparation of such a biography, he asks the assistance of members of the Society who may be able to loan him pertinent material. Such items as original letters, plans, reports, portraits, correspondence of contemporaries with Telford, or references in diaries and memoirs will be of great value. Any information as to possible sources of material will also be greatly appreciated. Members of the Society who can thus assist Sir Alexander Gibb should address him at Queen Anne's Lodge, Westminster, London, S.W. 1.

Helping the Young Engineer to Help Himself

Hints and Self-Analysis Blank Recommended by Committee on Professional Training to the Engineers' Council for Professional Development, Quoted in Full from the Council's First Annual Report

THE FIRST few years of his professional life present a sharp challenge to the young engineer. They are years in which he is likely to be relatively free from the insistent demands upon his time and energy which his family and his community will make later on. During these years, too, he is often learning the rudiments of business and his work will probably involve more routine and be less exacting and stimulating from an intellectual point of view than after he has gained the maturity and experience needed for more important tasks. To an unusual extent, therefore, the pace of his personal, intellectual, and social development at this time will depend on himself. He will establish, or fail to establish, habits of study and thought, of friendship, and of life that may well be the determining factors in his future.

It seems important, therefore, that the engineer develop a concrete plan for these few years, which will help him to use them in a truly constructive way. Such a plan should be based on the work which he is doing and is looking forward to, on his place in the engineering profession, and on his individual strength and weakness, his background and his ambition. The men who have drawn up the blank which follows believe that it will be helpful to those who are willing to use it thoughtfully and with honesty to themselves, as well as to carry out steadily a sound, balanced program which it may suggest.

In your use of it, they would suggest that you read it over carefully first and then ponder for a few days the questions which it raises in your mind. Particularly if some of those about your own aptitudes and handicaps puzzle you, you may find it helpful to discuss those points, with or without reference to the blank, with one or two intimate friends or some older man whose judgment and sympathetic understanding you trust. In formulating the program it suggests, it would be best not to make it too ambitious, that you may be able to do well the projects you undertake. It should, however, be substantial enough to make you feel that you are gaining knowledge, poise, and understanding as the months go by.

SELF-ANALYSIS BLANK FOR JUNIOR ENGINEERS

Occupation

1. What are the duties and responsibilities of my present job?

2. Have I the general education needed to do this job well? If not, what additional courses should I take, or what reading should I do?

3. Have I the knowledge of my job, department, and company needed to do it well? How can I gain additional knowledge, of value in it, through training courses, reading, or better use of my experience and a broader range of contacts?

4. Can I be depended on to carry out assignments given me promptly and thoroughly? If not, what is the difficulty, and how can I overcome it?

5. Am I a productive worker and is my work characterized by a professional finish? How can I improve or perfect the professional nature of all phases of my work?

6. (*a*) Are my relations with my superiors cordial and friendly? If not, what is there in my own attitude or lack of understanding of their responsibilities that may contribute to this, and how can I improve it?
(*b*) Are my relations with my associates cordial and friendly? If not, how does my own attitude contribute to this? Do I try to appropriate all the credit or shift the blame?
(*c*) Are my relations with my subordinates cordial and friendly? Are my instructions to them always clear and complete? Do I supervise properly to make sure that those instructions are understood, at the same time avoiding unnecessary interference in their execution? Do I give proper credit for good work, and do I insist on it? Am I genuinely interested in their ambitions and problems?
(*d*) Are my contacts with the customers of the company or other members of the public friendly and effective?
(*e*) Am I able to direct and mold all of these relationships toward the accomplishment of the purpose of the organization which I serve?

7. Am I alert in seeing ways by which the accomplishment of my work and that of the group of which I am a member can be improved? Do I think through my ideas for improvement carefully? If they still seem sound, am I ready to take the initiative in presenting them for consideration? Do I present my views to my superiors and associates in a clear, concise, and forceful manner?

8. What are my physical or emotional handicaps to the best performance of my work? Am I doing everything possible to overcome them?

9. What is the next position which I am anxious to secure? What are its duties and responsibilities?

10. What educational equipment, beyond that required in my present job, is needed to fill that position well, including both technical knowledge and that of economic or social trends? What courses or reading will enable me to acquire that equipment?

11. What knowledge of my particular company, beyond that required in my present job, is needed to fill that position well? Am I gaining that knowledge? If not, what should I do to secure it?

12. What personal qualities, beyond those required in my present work are needed to fill that position well? How can I develop those qualities more fully?

13. Is there any other job, in my own company or elsewhere, which I ultimately wish to secure and for which I should begin now to prepare myself? If so, what are

its duties and requirements, and what steps should I take to satisfy them?

14. Does the company with which I am now connected offer the ideals, associations, and opportunities for advancement which I can reasonably expect? If not, where should I seek a connection with genuinely greater promise in these respects?

PROFESSIONAL STATUS

1. What requirements for full membership in my professional society do I now fail to satisfy? What steps, beyond those already planned, should I take to meet them? When should I plan to attain such membership?

2. Do I possess the achievements which give eligibility for a professional degree in engineering? If not, what requirements do I need to satisfy eligibility? What efforts should I make to do so? Should I plan to secure such a degree?

3. Does my position and do my ambitions make it desirable for me to become a registered professional engineer? If I wish to do this, what steps should I take to meet the requirements? How soon should I plan to apply for registration?

PERSONAL STATUS

1. In terms of family relationships, what goals shall I set now? Are my education and cultural background adequate for their realization? If not, can I improve them by courses or reading in addition to those already proposed?

2. What physical and emotional shortcomings do I possess which tend to prevent their full realization? What steps should I take to overcome these handicaps?

3. Is my financial status adequate for their realization? If not, what program should I develop and try to carry out?

4. What place in my community do I wish to take? How can I prepare myself to contribute more to its civic life?

5. Am I developing my capacity for friendship and utilizing opportunities for broadening the circle of which I am a part? Do I contribute my share in the give-and-take of these relationships? How should I change my attitude or develop my personal qualities to deepen my friendships?

6. What avocation, recreation, sports, or hobbies would be best suited to round out my life?

GENERAL PROGRAM OF DEVELOPMENT

1. On the basis of this analysis, what educational courses and reading should I undertake during the next few years without undue stress on my time and physique?

2. What experience should I seek, and how can I best utilize it?

3. What personal qualities and attitudes should I cultivate?

4. What health program should I follow?

5. What financial program should I lay out and try to carry through?

6. What program of sport recreation or avocation should I plan?

IMMEDIATE PROGRAM

1. What definite steps in each of these directions should I take now?

NEWS OF ENGINEERS

From Correspondence and Society Files

FRED J. NEBIKER is now inspector of general construction in the U. S. Engineer Office at Jacksonville, Fla.

ROY M. HARRIS has resigned from the U. S. Coast and Geodetic Survey to accept the position of State Sanitary Engineer with the Washington State Department of Health. His headquarters are in Seattle, Wash.

JULIUS L. SPEERT is now employed as a junior topographic engineer in the Computing Section of the U. S. Geological Survey, in Washington, D.C.

W. S. BAVER has accepted a position as designing engineer in the Newark, N.J., Division of Sewers. He was formerly employed in a similar capacity by Kenneth Franzheim, of New York, N.Y.

PERCY H. BLISS recently severed his connection with the U. S. Coast and Geodetic Survey to accept a position as instructor in civil engineering at the Oregon Institute of Technology, at Portland, Ore.

JOHN F. HAGERTY, formerly employed by the Inland Steel Company, of Chicago, Ill., is now serving as engineer inspector for the Seattle Municipal Street Railways, with headquarters in Seattle, Wash.

ROBERT J. ALPHER has taken a position as structural engineer in the Bureau of Agricultural Engineering of the U. S. Department of Agriculture, in Washington, D.C.

PHILIP H. CARLIN has joined the staff of C. E. Myers, consulting engineer, of Philadelphia, Pa.

GILBERT H. DUNSTAN has been promoted from the position of instructor in civil engineering at the University of Southern California to that of assistant professor of general engineering.

GUNNI JEPPESEN is now bridge engineer in the State of Illinois Division of Waterways, with headquarters in Joliet, Ill.

JAMES LOGAN has resigned as secretary and treasurer of the Hill Construction Company, of Mount Holly, N.J., to serve as Assistant State Highway Engineer of New Jersey. His offices are in Trenton, N.J.

SAMUEL M. RUDDER has been promoted from the position of division engineer for the Missouri State Highway Department to that of engineer of surveys and plans in the same department. His headquarters will continue to be Jefferson City, Mo.

G. J. KENNEDY has been promoted from the position of assistant general manager of the Philippine Railway Company to that of vice-president and general manager of the same organization. His headquarters are Iloilo, P.I.

W. G. BRENNEKE is now engaged by the CWA as chief of party on a surveying project of the U.S. Coast and Geodetic Survey in the Hartford, Conn., district.

JOHN G. ENGLISH recently joined the staff of the P. T. Cox Contracting Company, of New York, N.Y., in the capacity of superintendent of construction.

DEAN G. EDWARDS has been made chief engineer of the Civil Works Administration of New York, N.Y. He was formerly president of Edwards and Flood, Inc., of Brooklyn, N.Y.

WENDELL L. FOSTER has taken the position of county engineer of Woods County, Oklahoma. His office is in Alva, Okla.

JOHN L. NEELY, JR., who is in the employ of the Tennessee Valley Authority, has recently been transferred from Knoxville, Tenn., to Nitrate Plant, Ala., where he is manager of Muscle Shoals Plants.

A. W. YEREANCE has been promoted from the position of regional appraiser in the Real Estate Loan Department of the Prudential Insurance Company to that of assistant manager of the Boston branch office of the Mortgage Loan Department of the same organization.

ALBERT HARRISON HINKLE, formerly chief engineer of maintenance of the Indiana State Highway Commission, is now State Engineer of Indiana for the Federal Emergency Administration of Public Works. His offices are in Indianapolis, Ind.

PHILLIPS MOORE has resigned as transitman and assistant resident engineer of the Georgia State Highway Department to accept an appointment as State Airport Engineer with the Aeronautics Branch of the U. S. Department of Commerce.

FREDERIK KRUYS is now Government Engineer in the Bridge Department of the Rykswaterstaat, The Hague.

HAROLD E. WESSMAN has resigned as superintendent of the Water Department of Rockford, Ill., to become Associate Professor of Mechanics and Structural Engineering at the State University of Iowa, Iowa City, Iowa.

W. R. CHAWNER, formerly commercial engineer of the Southern Sierras Power Company, of Riverside, Calif., has now joined the staff of the Temescal Water Company, of Corona, Calif., in the capacity of chief engineer and general manager.

FREDERICK J. FRICKE, who was formerly employed as a junior engineer in the topographical branch of the U. S. Geological Survey, has now joined the staff of the U. S. Army Engineers in the capacity of junior civil engineer. His headquarters are Galveston, Tex.

HERMAN A. BLAU has taken the position of assistant bridge construction engineer on the construction of the San Francisco-Oakland Bay Bridge, with offices in San Francisco, Calif.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From March 10 to April 9, 1934, Inclusive

Additions to Membership

Aalto, Johan August (Jun. '34), Ardsley, N.Y.

Adams, Walter Kelsey (Assoc. M. '32), With E. P. Arneson; 837 West Woodlawn Ave., San Antonio, Tex.

Albert, James George (Jun. '34), 127 Liberty St., New York, N.Y.

Albertis, Alexander John (Jun. '32), With N. G. Camilos & Co., Capodistriou 27, Athens, Greece.

Baker, John Fleetwood (Assoc. M. '34), Prof., Civ. Eng., Univ. of Bristol; 1 Tyndall Ave., Bristol, England.

Beatty, Richard Vilas (Jun. '33), 1229 Delia Ave., Akron, Ohio.

Brown, Charles Reed (Jun. '34), Chf. of Party, U. S. Engrs., U. S. Engr. Office, Glasgow, Mont.

Bundy, Willard Lindley (Jun. '33), Crossville, Tenn.

Burland, Carlyle Gray (Jun. '34), Chf., Survey Party, State Dept. of Public Works (Res., 18 Cottage Pl.), Leominster, Mass.

Campbell, Ray Anderson (Jun. '34), 704 South David St., Casper, Wyo.

Castellucci, Joseph (Jun. '33), 184 Cumberland St., Providence, R.I.

Church, James Duncan, Jr., (Jun. '33), Asst. Engr., U. S. Bureau of Reclamation, Denver, Colo.

Claus, Theodore Otto (Jun. '33), 3987 Maybury Garden, Detroit, Mich.

Deming, Stephen Arthur (Jun. '34), Asst. Project Engr., State Highway Dept. (Res., 4744 Bellevue Ave.), Kansas City, Mo.

Doyle, William Christian (Jun. '33), Mass. Inst. Tech. Dormitories, Box 195, Cambridge, Mass.

Easterbrooks, Preston Burt, Jr., (Jun. '34), 2 Goodwin Pl., Boston, Mass.

Gard, Walter Sumner (Jun. '34), 100 Michigan Ave., Charleston, W.Va.

Gardiner, Arthur Perry (Jun. '33), 1760 Pennsylvania, Denver, Colo.

Gardner, Robert Anthony (Jun. '33), Instr., Bucknell Univ., Lewisburg, Pa.

Hauger, Roy Leo (Jun. '33), 215 Muncey St., San Antonio, Tex.

Holland, Elwood William (Jun. '34), Jun. Engr., Met. Water Dist. of Southern Calif.; Box 381, Banning, Calif.

Humes, John Donald (Jun. '33), 3008 Harvard North, No. 107, Seattle, Wash.

Imperiale, Michael Aloysius (Jun. '33), 615 Union St., Brooklyn, N.Y.

Johnson, Cecil William (Jun. '33), 1119 East 43d St., Seattle, Wash.

Kane, John Edmund (Jun. '33), 857 Bailey Ave., Elizabeth, N.J.

Kempton, Ray Addison (M. '34), 3 Elliot Rd., Great Neck, N.Y.

Klingenberg, Albert Charles (Assoc. M. '34) (Catalano Constr. Co.), 2834 Mayfield Ave., Baltimore, Md.

Koenig, John William (Jun. '33), 735 Sedam St., Cincinnati, Ohio.

Leithmann, Warren Justin, Jr. (Jun. '33), 659 Gerhart St., Roxborough, Philadelphia, Pa.

Lewis, Richard Gaston (Jun. '33), 410 North 2d Ave., Hopewell, Va.

Lin, Tung Yen (Jun. '33), With Eng. Dept., Tientsin-Pukow Ry.; 32 Mien Hsieh Ying, Nanking, China.

McKeon, William Thomas (Jun. '33), 92 Sterrett St., Crafton, Pa.

McNamara, Stephen James (Assoc. M. '34), Asst. Chf. Estimating and Designing Engr., Grip Steel Bar Co., Ltds., Princess St. (Res., 5 Lichfield Drive, Sedgley Park, Prestwich), Manchester, England.

Maess, Albert Charles (Jun. '33), Ballston Lake, N.Y.

Middleton, Robert Alfred (Jun. '33), 4515 Fourth Ave., N.E., Seattle, Wash.

Moritz, Gordon Durrie (Jun. '33), Raritan Ave., Bound Brook, N.J.

Morris, Edgar Longette (Jun. '33), 5102 Ninth St., N.W., Washington, D.C.

Nardelli, Dante (Jun. '33), 429 Charles St., Providence, R.I.

Narver, David Lee (M. '34), (Holmes & Narver, Inc.), 639 South Spring St., Los Angeles, Calif.

Ogden, Horatio Nash (Jun. '33), 910 South Carrollton Ave., Apartment C, New Orleans, La.

Papandrea, Natale Neo (Jun. '33), 57 Smith St., Newark, N.J.

Pegado, Henrique (M. '33), Engr. (Henrique Pegado & Cia, Ltd.); Gen. Mgr., Cia. Suburbana Industrial; Prof., Mathematics, Mackenzie Coll. (Res., Rua Libero Badaro 47-4° andar), Sao Paulo, Brazil.

Pilkington, George Brown (Jun. '33), 221 Mulberry St., Marianna, Ark.

Pitzen, Tracy Allen (Jun. '33), 209 Brockwell Arms, Waterloo, Iowa.

Procuniar, Robert William (Jun. '34), 1316 Highland Ave., Dayton, Ohio.

Reyes, Rafael (Jun. '33), 31 Mount Hope Pl., New York, N.Y.

Ronka, Arne Henry (Assoc. M. '33), 76 Langsford St., Gloucester, Mass.

Rover, Henry John (Assoc. M. '34), Asst. Engr., Dept. of Public Works, Borough of Manhattan; 80 Winthrop St., Brooklyn, N.Y.

Salisbury, Eugene Franklin (M. '34), Chf. Engr., L. & A. Ry., Minden, La.

Smallwood, David Montgomery (Jun. '33), 2316 Tasker St., Philadelphia, Pa.

Sommer, Matthew Michael (Jun. '33), 3818 North Franklin St., Philadelphia, Pa.

Sparling, Jack Norman (Jun. '33), 737 South Adams St., Glendale, Calif.

Sulkowski, Walter Valentine (Jun. '33), 546 Hertel Ave., Buffalo, N.Y.

Taylor, Alva Almond (Assoc. M. '34), Asst. City Engr., 2120 Cherry Ave. (Res., 3312 Lemon Ave.), Signal Hill, Calif.

Van Hasselt, Jacob Adriaan Karel (M. '34), Prof. of Civ. Eng., Antioch Coll., Yellow Springs, Ohio.

Van Wingen, Nico (Jun. '34), 1021 Coronado Ave., Long Beach, Calif.

Weirick, Fritz Henry (Jun. '34), 605 West 8th St., Topeka, Kans.

Willis, Lovell (Jun. '33), With Sperry & Treat Co., 294 Kimberly Ave. (Res., 131 Westwood Rd.), New Haven, Conn.

Winsmore, Karl Buhre (Jun. '33), 108 Park Ave., Ridley Park, Pa.

Woodruff, John Dewey (Jun. '33), 1611 Central Ave., Dodge City, Kans.

Young, Joe Wah (Assoc. M. '34), Engr., Palmer & Turner, 1 Canton Road, Shanghai, China.

Membership Transfers

Bell, Edward Arthur (Jun. '28; Assoc. M. '34), Engr. and Surv., 133 Lake Ave., Boonton, N.J.

Hou, Chia Yuen (Jun. '21; Assoc. M. '28; M. '34), Director, Dept. of Public Works, Nanking Municipality, Nanking, China.

Jacobs, Harry Victor (Jun. '30; Assoc. M. '33), Res. Engr. (W. N. Brown, Inc.), 969 National Press Bldg., Washington, D.C.

Kiehnle, William Albert (Jun. '23; Assoc. M. '28; M. '34), Engr. and Secy. (William V. Kiehnle Co.), 3606 Park Ave., New York, N.Y.

Little, Charles Rex (Jun. '28; Assoc. M. '34), 175 Stonewall St., Apartment 304, Memphis, Tenn.

Otter, John Vernon (Jun. '30; Assoc. M. '34), 403 North Polk St., Moscow, Idaho.

Ponco, Jacinto Samonte (Jun. '30; Assoc. M. '34), Asst. Civ. Engr., Bureau of Public Works, Manila, Philippine Islands.

Rohlwing, Anson William (Jun. '26; Assoc. M. '33), Field Engr., Portland Cement Assoc., Chicago, Ill. (Res., 1859 Princeton Drive, Louisville, Ky.)

Veranth, Joseph (Jun. '30; Assoc. M. '34), City Engr. (Res., 128 East Pattison St.), Ely, Minn.

Wickham, Joseph John (Jun. '27; Assoc. M. '34), Text Book Writer, Civ. Eng.; Instr., Pennsylvania State Coll. Extension School (Res., 735 Harrison Ave.), Scranton, Pa.

Reinstatements

Armstrong, George Simpson, Assoc. M., reinstated March 26, 1934.

Cummins, Charles Albert, Assoc. M., reinstated March 16, 1934.

Resignations

Bowden, Foster Sidney, Jun., resigned March 21, 1934.

Chapin, Charles Walter, Assoc. M., resigned March 21, 1934.

Kiehm, Charles, Assoc. M., resigned April 5, 1934.

Major, Samuel Robertson, Jun., resigned April 5, 1934.

Moncur, George, M., resigned March 23, 1934.

Quade, Maurice Northrop, Assoc. M., resigned March 21, 1934.

Deaths

Brown, Earl Clarence. Elected Assoc. M., Aug. 27, 1928; died March 27, 1934.

Churchill, Charles Samuel. Elected M., May 1, 1889; died Jan. 25, 1934.

Courtenay, William Howard. Elected M., July 3, 1889; died March 15, 1934.

Delaplaine, Henry. Elected M., Jan. 17, 1927; died March 13, 1934.

Eaton, Frederick. Elected M., May 5, 1886; died March 9, 1934.

TOTAL MEMBERSHIP AS OF APRIL 9, 1934

Members	5,753
Associate Members	6,271
Corporate Members	12,024
Honorary Members	18
Juniors	3,137
Affiliates	108
Fellows	4
Total	15,291

FREYHOLD, FELIX. Elected Assoc. M., Sept. 2, 1891; M., April 2, 1902; died Feb. 15, 1934.

GREEN, SAMUEL MARTIN. Elected M., July 1, 1908; died March 22, 1934.

HILLMAN, GEORGE WALDO. Elected Assoc. M., April 1, 1908; died Nov. 11, 1933.

JOHNSON, GEORGE ARTHUR. Elected Assoc. M., Feb. 6, 1907; M., July 2, 1913; died April 1, 1934.

LEE, WILLIAM STATES. Elected M., Oct. 4, 1905; died March 24, 1934.

MATAMOROS, LUIS. Elected M., Oct. 4, 1905; died Jan. 18, 1934.

NEWMAN, WILLIAM ARNOLD. Elected Assoc. M., March 15, 1926; died Oct. 30, 1933.

URSDELL, JOHN ARNOLD. Elected M., June 3, 1908; died May 15, 1933.

VAN CLEVE, AARON HOWELL. Elected Jun., May 31, 1892; Assoc. M., Dec. 5, 1894; M., Nov. 1, 1904; died March 31, 1934.

WATSON, ROBERT MALCOLM. Elected M., April 18, 1916; died March 4, 1934.

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 85 of the 1934 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York Office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

Construction

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 31; single; B.S. in C.E. (cum laude) Harvard University; 5½ years field and office engineer with contractor on subway construction, in New York City, involving also underpinning of buildings and reconstruction of sewers, utility mains, and ducts. Location anywhere in the United States. Best references. D-2878.

HIGHWAY AND MUNICIPAL ENGINEER; Jun. Am. Soc. C.E.; 29; married; graduate; registered civil engineer; 7 years experience on highway design and construction; 2 years in municipal work, including city pavements, sewers, and water supply and distribution. Leaving present position May 1st due to curtailment of activities. C-5490.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; B.S. in C.E., 1929; age 28; married; successfully completed one year of law; experience in general construction, field and office; estimating; supervising; surveying; good draftsman and computer; licensed engineer in New Jersey; desires position with chance for reasonable advancement; available immediately. D-198.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; graduate of New York University; married; 2½ years experience in topographic surveying, route survey, and building construction as chief of party and instrumentman; 1 year as job clerk and estimator in building construction. Desires position in field. Available immediately. D-379.

CIVIL ENGINEER AND CONSTRUCTION SUPERINTENDENT; Jun. Am. Soc. C.E.; 30; married; New York University, 1924; 4 years experience in responsible charge of property, topographic, road and pipe-line surveys, also triangulation, including necessary computations; 5 years experience in charge of road, sewer, water, and underground utility construction. B-6472.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 36; bachelor's, master's, and professional degrees, Purdue University. Experienced in concrete girder, arch, and steel bridges as: inspector, field and office engineer, draftsman, designer, field superintendent of erection; designer and detailer of concrete forms in panels, cofferdams, arch centering, falsework, erection equipment, plant layouts. D-864.

Design

COLLEGE GRADUATE; Assoc. M. Am. Soc. C.E.; licensed professional engineer, with 19 years experience on design, estimate, and alteration of industrial buildings, offers his services. C-4607.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; two college degrees; 23 years experience in designing, surveying, and estimating for contractor. Factories, railroads, housing, real estate development. A-2505.

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 30; B.S. in C.E. and C.E. degrees; 5 years experience as a designer, draftsman, checker, detailer, and estimator of structural steel and reinforced concrete structures. References and samples of work sent on request. C-6186.

STRUCTURAL DESIGNER OR DRAFTSMAN; Jun. Am. Soc. C.E.; graduate of Polytechnic Institute of Brooklyn, C.E. degree; 28; single; willing to travel; 6 years drafting, design, checking, and estimating structural steel and reinforced concrete for subways, buildings, and steam and hydro-electric power plants and transmission lines; 3 years mapping and estimating underground conduit systems. C-9972.

Executive

CIVIL AND SANITARY ENGINEER; Assoc. M. Am. Soc. C.E.; experienced in water supply, water purification, and sewage treatment problems. Varied experienced in design and testing of concrete and in manufacture of concrete pipe and piles. Construction experience on port works. Comprehensive experience in hydrographic surveys. Married. Wants job offering some future. D-2889.

PROJECT ENGINEER; MECHANICAL AND CIVIL; M. Am. Soc. C.E.; 41; industrial, power, and railway plants; buildings, equipment, and facilities. General and detailed design, construction, and remodeling. Experienced charge technical division. Cooperation research departments on plant development. Some sales promotion; 20 years experience in United States and South America. Speaks Spanish. D-2887.

ENGINEER; Assoc. M. Am. Soc. C.E.; degree E.M. at Lehigh University, 1908; 15 years experience, including engineering supervision of plant construction, railroads, pipe-line, transmission-line construction; also mining and metallurgical construction, tests, operation, and metal mine examinations. Broad surveying experience in connection with this work. C-2245.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 51; graduate; 28 years responsible charge of construction and design of municipal water-supply structures, locks, dams, reservoirs, river improvement, bridges, shafts, vehicular tunnel, irrigation canals and structures, etc. Qualified for an important position. Location and salary open. B-5412.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; licensed in Pennsylvania; 30; married; Cornell graduate; continuous employment with one company since graduation; over 4 years as office engineer; 2 years as field engineer; 2 years in design; experience covers industrial work, power plants, railroads, office, store, and apartment buildings. D-2891.

EXECUTIVE ENGINEER; Assoc. M. Am. Soc. C.E.: seeks connection with financial house or engineering concern doing domestic or foreign business. C-9867.

STRUCTURAL ENGINEER EXECUTIVE; M. Am. Soc. C.E.; 46; married. Connected with a leading steel fabricating business for over 20 years, engineering, operating, and sales. Qualified to handle any responsible position most efficiently. Available immediately. C-5095.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; licensed; graduate of Cornell University, 1919; years of experience in the following: sewer construction, highway maintenance and design of steel and reinforced concrete, bridges, buildings, bulkhead piers, and hydraulic structures; wishes to connect with an engineering firm. A-4279.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; B.S., M.S., Ph.D.; 7 years experience on design and construction of steel and concrete structures. Available immediately. C-3736.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; American; 47; married; 28 years experience; 9½ years drafting, design; 18½ years field, construction, surveys. Principal engagements, water supply, railroad, hydro-electric, mill work. Good record with former employers, having had four return engagements. Will consider position in any capacity mentioned or in any locality with reputable organization. B-5199.

Junior

RECENT GRADUATE; B.S. in C.E.; Jun. Am. Soc. C.E.; 24, single; brief experience as levelman with U. S. Coast and Geodetic Survey; actual experience in quarrying, inspecting, and operating hoisting engines in granite quarry; speaks Italian; desires any connection in any capacity where resourcefulness and willingness are essential. Location immaterial. D-2871.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; C.E. degree, Polytechnic Institute of Brooklyn: 9 months surveying experience, transitman and chief of party; 4½ years experience drafting, computing, estimating on subway construction. Desires any job on construction, either field or office. Further experience paramount. Salary secondary. Location immaterial. Can start immediately. C-3168.

JUNIOR CIVIL; Jun. Am. Soc. C.E.; 4 years experience transitman, maps, computations on highway, waterway, and underground projects. Record includes New York Central and U. S. Gypsum Company assignments. Qualified for Junior Engineer in U. S. Coast and Geodetic Survey. Single; citizen; age 30; able-bodied; good eyesight. Will go anywhere except tropic lowlands. C-6530.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; A.B., 1930, Dartmouth; C.E., 1931, Thayer School of C.E. Experience surveying on building construction jobs and in field, design of simple steel and reinforced concrete structures, drafting, etc. Desires position in any branch of civil engineering. Salary and location secondary. Available immediately. D-2696.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; B.C.E., cum laude, Polytechnic Institute of Brooklyn, 1933. Majored in advanced bridge design. Excellent mathematician; competent draftsman and surveyor; 6 months experience, drafting and as instrumentman. Desires opportunity in any branch of civil engineering, field or office. Salary and location secondary. Available immediately. D-2913.